Fay, Spofford & Thorndike, Inc.
Engineers
Boston, Mass.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS

CAPE COD CANAL
BOURNE HIGHWAY BRIDGE

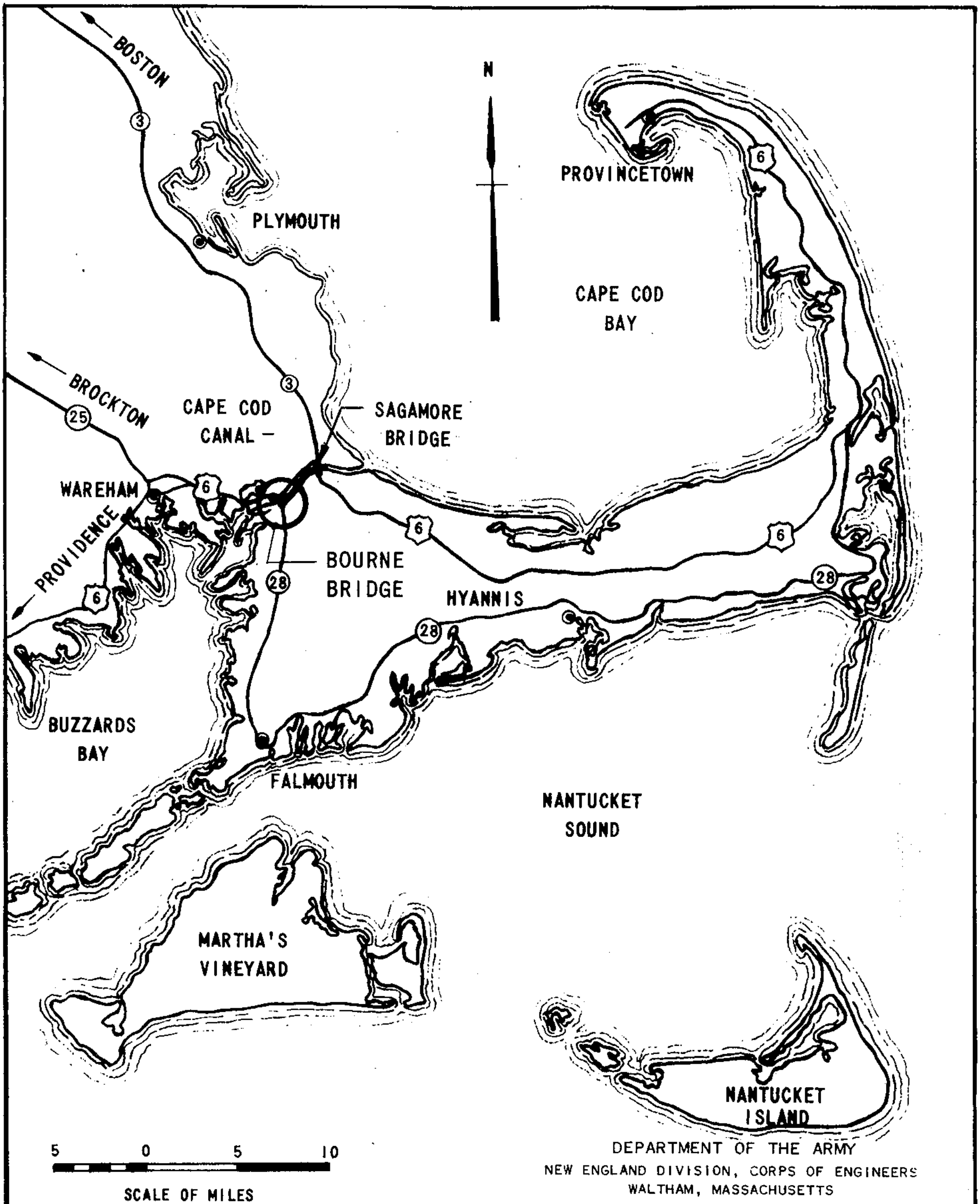
1971 CONDITION REPORT

CONTRACT NO. DACW33-71-C-0105

Fay, Spofford & Thorndike, Inc.
Engineers
11 Beacon Street
Boston, Massachusetts

September, 1971

COMMONWEALTH OF MASSACHUSETTS
RICHARD
W.
ALBRECHT
REGISTERED
PROFESSIONAL ENGINEER



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

CAPE COD CANAL
BOURNE, MASS.

BOURNE HIGHWAY BRIDGE
1971 CONDITION REPORT

LOCATION PLAN

FAY, SPOFFORD & THORNDIKE, INC., BOSTON, MASS.

FAY, SPOFFORD & THORNDIKE, INC.
ENGINEERS
BOSTON, MASS.

I N D E X

PLATES

FAY, SPOFFORD & THORNDIKE, INC.
ENGINEERS
11 BEACON STREET • BOSTON, MASSACHUSETTS 02108
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September 30, 1971

Department of the Army
New England Division
Corps of Engineers
424 Trapelo Road
Waltham, Massachusetts 02154

Subject: Contract No. DACW33-71-C-0105
Inspection and Condition Report
Bourne Highway Bridge
Cape Cod Canal - Massachusetts

Gentlemen:

We are pleased to present herewith our report of the field inspection and our conclusions concerning the condition of the Bourne Highway Bridge crossing the Cape Cod Canal. This bridge consists of a three span continuous steel structure over the canal, flanked by deck type steel trusses with concrete abutments and piers. The bridge carries four lanes of traffic and a single sidewalk.

The general condition of the bridge is good to excellent. Such deterioration as has been noted is in localized areas but is not of a nature that reduces the structural adequacy of the bridge. The corrosion is most severe in certain areas, particularly beneath the expansion joints, where runoff from the roadway containing deicing chemicals has flowed across the structural portions. The expansion joints should be repaired, preferably by replacement with a more modern and efficient type, to prevent further deterioration of this nature. Some of the secondary parts of the structure, such as lacing on certain members, has also deteriorated and should be repaired. A relatively small number of rivets show corrosion severe enough to require replacement. A careful inspection, including X-rays, of the supporting cables indicates no corrosion. However, a possibility of internal deterioration of the cables or deterioration within the end Pg. 59

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ENGINEERS
BOSTON, MASS.

anchorages cannot be eliminated completely without removal of and destructive testing and inspection of the cables. Consideration should be given to such a procedure, as these cables are critical from a safety standpoint.

Cracks in the concrete of the piers and abutments have been repaired recently. These repairs have been effective, and these parts of the structure are, with minor exception, in excellent condition. The only surface deterioration of the concrete observed was localized and of no structural significance. Minor repairs to the concrete should be made, as indicated in the report.

Immediate action should be taken with regard to the replacement of the anchor bolts for the lamp posts, as these have deteriorated to such an extent that there is a danger that some of the posts may fall under adverse weather conditions. Similarly, a localized area of spalled concrete on Pier 4 should be repaired, as a large piece of concrete may break loose at any time.

The design analysis of the capacity of the structure to carry current loadings indicates that any possible traffic load is well within the capacity of the structure. However, the current design loads of the American Association of State Highway Officials for wind has been increased 66 percent over the wind loads originally used for the design of this bridge. This results in theoretical overstress in certain members loaded by wind alone. Considering the conservativeness of current wind load, we do not believe the overstress that may occur will be of significance. With regard to the capability of this bridge to handle current highway traffic safely, it should be noted that the widths of the traffic lanes are less than is now considered desirable and opposing traffic is not physically separated.

As requested, we have included an estimate of cost to make the recommended repairs. We find that slightly more than $500,000 will be necessary for this purpose.

We are pleased to have had the opportunity to make this study and wish to acknowledge the excellent cooperation of your staff in furnishing data to us and to thank the Department of Public Works of the Commonwealth of Massachusetts for the traffic data which they supplied.

Very truly yours,

FAY, SPOFFORD & THORNDIKE, INC.
By *Richard W. Albrecht*

Richard W. Albrecht,
Director

RICHARD W. ALBRECHT

RWA:vj

REPORT

GENERAL HISTORY - CAPE COD CANAL BRIDGES

In 1697, the General Court of Massachusetts ordered a feasibility report for the construction of a canal from Buzzards Bay to Cape Cod Bay. There is no record of this report ever having been made or submitted through the General Court but it does indicate a very early interest in a canal at this location.

In 1899, the Massachusetts Legislature granted a charter to the Boston, Cape Cod and New York Canal Company, for the construction of a canal. Work began in 1909 and was completed in April, 1916. The project dimensions of the canal constructed were 25 feet depth, with a bottom width of 100 feet, widening to 300 feet at the east end and to 250 feet at the west end. The canal was eight miles long in land cut and had dredged approaches of one-half mile in Cape Cod Bay and five miles in Buzzards Bay.

Because of the difference in phase and height of tides at the two ends of the canal, its narrow width and fast currents, traffic using the canal began to decline in 1919.

The United States Government purchased the canal in 1928 at a cost of $11,500,000 and placed it under the supervision of the United States Army, Corps of Engineers.

The original bridges crossing the canal consisted of two highway bridges and one railroad bridge. All were single span bascule structures with clear distances between fenders of 140 feet. The original highway bridges carried an average of 20,000 vehicles on Sundays during the summer, with a reported peak traffic of 40,000 vehicles on the day of the eclipse in 1932. In 1932, the United States Army, Corps of Engineers, recommended ultimate widening of the canal to 500 feet bottom width with a 40 feet depth. The National Industrial Recovery Act of 1933 authorized the expenditure of $23,500,000 for this purpose. On August 31, 1933, the War Department allocated $4,600,000 for the construction of three bridges over the widened canal.

The criteria for the new highway bridges required a vertical clearance of 135 feet above high water and a span that would clear the newly proposed channel limits.

The firm of Fay, Spofford & Thorndike was engaged as designing and supervising engineers for the two highway bridges. A construction contract for the bridge foundations was awarded to P. J. Carlin Construction Company of New York on December 8, 1933. A contract for the construction of the superstructure was awarded to the American Bridge Company on January 25, 1934.

The low banks of the canal, and the high clearances and substantial lengths of the bridges required, dictated the selection of three span continuous trusses, partially deck and partially through. This construction made it possible to use the same main structure for both the Sagamore Bridge and the Bourne Bridge. As high ground is relatively close to the canal at the Sagamore Bridge, it was possible to carry the embankment up to the ends of the main bridge structure. At the Bourne Bridge additional truss spans were required at each end.

The bridges were opened to traffic on June 22, 1935, with a formal dedication in August, 1935.

<u>Bourne Bridge</u>

The Bourne highway bridge consists of seven spans. Simple truss spans of 240 feet and 270 feet on the south side and 240 feet and 208 feet on the north side, flank the three continuous channel spans. The substructure consists of the two channel piers, four intermediate piers and two abutments. The total bridge length between centerlines of abutment bearings is 2,384 feet. Each abutment provides a concrete framed bridge deck 150 feet long, making the total structure length 2,684 feet.

The roadway is suspended from the truss by double cables at each end of each floorbeam for the middle two-thirds of the main channel span. The floorbeams of the side spans and the portions of the main span immediately adjacent to the piers are framed directly into the trusses.

The majority of the truss members and the floor beam flanges of all spans were constructed of high strength silicon steel. The stringers of Spans 1, 2 and 3 are also constructed of silicon steel. The roadway deck was constructed of lightweight (Haydite) concrete. This construction resulted in a lighter superstructure with consequent savings in steel and improvement in appearance.

HISTORY OF MAINTENANCE, REPAIRS AND ALTERATIONS
(Bridge Open to Traffic - June, 1935)

Painting Record

Fiscal Year 1938
" " 1947
" " 1951 & 1952
" " 1958
" " 1967
" " 1971 (Railings only)

Paving Record

Fiscal Year 1938 (Seal coat of sheet asphalt)
" " 1949
" " 1964 (Part of 1963 Major Rehabilitation)

REPAIRS AND ALTERATIONS

Fiscal Year	Drawing Nos.	Drawing Titles	Description of Work
1937	Maintenance		Repair hole in deck - Span 1
1955	Maintenance		Repair hole in deck - Span 4
1959	Feb., 1959 F1112-E-1-1 (One Sheet)	Repairs to Bourne & Sagamore Bridges	Replaced four anchor bolts - 1 each bearing - Piers 3 and 5
1960			Repaired spalled concrete on Pier 4
1962	Maintenance		Replaced sections of underdeck cat-walk grating
1962			Installation of ladder safety devices

Fiscal Year	Drawing Nos.	Drawing Titles	Description of Work
1962	Maintenance		Installed covers over roller bearings
1963	Maintenance		Repair hole in deck - Span 3
1963	VL-15B (One Sheet)	10-inch Gas Main Bourne Bridge	Ten inch diameter welded steel gas main installed between Piers 3 and 4 with risers at piers. (Work for and by Buzzards Bay Gas Co.)
1963	1156-E-1-1 (Ten Sheets)	Bourne Highway Bridge - Major Rehabilitation	Additional access ladders and platforms, downspouts added to scuppers, replace catwalk grating, resurface roadway, sidewalk and curb, new curbing, rebuild expansion joints, replace 5-foot strips of deck adjacent to curbs, electrical work, new scuppers, repairs to concrete deck
1969	May, 1969 CAP-11 (Six Sheets)	Sagamore and Bourne Highway Bridges Structural Repairs	Pressure grouting of cracks in abutments and piers

Annual Maintenance

(1) Cleaning, greasing and aligning roller bearings.

(2) Patching concrete curbs and sidewalks. This maintenance was accomplished prior to 1964 when granite curbing and sidewalk waterproofing were installed.

(3) Patching bituminous concrete roadway paving.

(4) Spot painting of structural steel.

(5) Repair to expansion joints (welding and/or bolting plates and placing plastic compound).

Shop Drawings

American Bridge Co.	Order No.	Sheet No.
Erection Procedure	G5228	767-1
Jacking Arrangement	G5228	1-9
Railing	G5229	1-3
Bearings	G5230	E1, 25-27
	G5234	52-57
	G5235	65
	G5236	E102
Truss Members	G5230	1-17, 30-33
	G5231	1-12
	G5234	9-39
	G5235	1-27
	G5236	102
Bracing Members	G5234	47-51, 58-66, 74, 78
	G5235	33-50
Floor Beams	G5230	21-24
	G5231	22-29
	G5234	1-6, 73-76
	G5235	28-31
Stringers	G5230	18-20
	G5231	19-20
	G5234	40-46, 63
	G5235	53-59, 61-64, 69
Fascia Beams	G5234	72
	G5235	60
Fence Posts	G5230	28-29
	G5234	E11, 67-71, 77
	G5235	32, 51-52
Lamp Posts	G5234	7-8, 79-81

Shop Drawings

		Sheet No.
Kalman Steel Co.		
Welded Truss Mats, Road Deck	No. 34-5	K1 to K6
P.J. Carlin Construction Co.		
Pier reinforcement, granite facing, abutment foundation, reinforcement, deck, walls, pylons.	No. BR-65	1-110
Phoenix Bridge Co.		
Pier reinforcement	No. 327	1-8

These drawings, in addition to the original contract drawings, are on file at the following locations:

Department of the Army
New England Division, Corps of Engineers
Cape Cod Canal Office
Buzzards Bay, Massachusetts

Department of the Army
New England Division, Corps of Engineers
424 Trapelo Road
Waltham, Massachusetts

Fay, Spofford & Thorndike, Inc.
11 Beacon Street
Boston, Massachusetts

INSPECTION PROCEDURES

An ironworker crew was utilized during the inspection to provide rigging where required and to assist in the inspection of the areas where difficult climbing was required for access to certain parts of the structure. The crew consisted of four journeymen ironworkers and a foreman, provided by the steel erection firm of Owen J. McGarrahan Co. of Cambridge, Massachusetts.

The engineers started the field inspection three weeks prior to the arrival of the ironworker crew. This time was utilized on the review of foundation data, substructure inspection and the inspection of parts of the superstructure readily accessible. A preliminary inspection of the superstructure was completed during this period to determine which areas would require special attention.

During the fourth week of the field inspection, the ironworker crew started work and the detailed inspection of the superstructure was started. Equipment used to facilitate access to the structure included a two place powered personnel basket. Two teams, each consisting of two ironworkers and one engineer were employed in the detailed inspection. These teams covered the entire superstructure inspecting every joint and member. A third team consisting of two engineers made detailed inspections of less exposed areas.

The inspection work was coordinated with government contractors working on the bridge and with Corps of Engineers personnel who were carrying out regular maintenance operations. The Corps of Engineers scheduled the annual maintenance of bearings to coincide with the inspection to avoid duplication of the work of removing plates and cleaning the rollers. With the assistance of the iron workers, the engineers were able to reach all parts of the structure for visual inspection of any deterioration found by any member of a team. Areas of the underdeck where the upper chord horizontal bracing consists of upturned angles had to be observed from the vertical truss members.

Numerous photographs were taken showing both the general condition of the various members and parts of the structure and details of the deteriorated areas found. The clearest and most definitive of these photographs were selected and included as Appendix I to this report.

The ironworker crew was also utilized to provide rigging for X-Ray technicians and to remove plates from the truss bearings for inspection.

The procedures utilized for the various instrument surveys will be described in the sections of the report dealing with these surveys.

Fay, Spofford & Thorndike personnel engaged in the field inspection and preparation of this report were as follows:

Richard W. Albrecht - Project Manager

Robert L. Harrington - Project Engineer

Marcus Hann - Engineer

Isaac Pargman - Engineer

Robert E. Bertolino - Engineer

Because of the size of this bridge and the numerous members which had to be identified in the taking of field notes and during the writing of this report, a joint numbering system was devised. The system employed is an extension of that used on the original design drawings. This system together with pier and span number designations is shown on the Key Plan included as plate I of this report. The legend given on this drawing is self explanatory and is used throughout the report to identify the part of the structure being discussed. The abbreviations shown in the legend also apply to the abbreviations used in the "Summary of Inspection Notes". In the photograph books of Appendix I, the second picture number, in parenthesis, is the roll and negative number. Field Book II C gives the field book reference of all the photographs taken.

REVIEW OF FOUNDATION DATA

All available foundation data was reviewed to determine the probable condition of the bridge foundations. The data reviewed are as follows:

1. Substructure Contract Plans
   Sheet 5 of 19
   "Bourne Bridge - Record of Borings"
   (Included as Plate II of this report).
2. Drawing No. DS20-2F
   (Two Sheets)
   "Sagamore and Bourne Bridge Survey Data"
   (Data for years 1935 to 1970)
3. Drawing No. File 472-1 E-12-1 (DS13/F1-1)
   (One Sheet)
   "Bourne Bridge Foundations As Built,
   Piers 1, 2, & 6"
4. Bourne Bridge Maintenance Sections
   (Twelve Sheets)
   Years 1957 through 1969

The foundations of the Bourne Bridge have been checked for movement and settlement at regular intervals since initial construction, by surveys conducted by Corps of Engineers personnel. These data are presented by the two drawings listed under Item 2 above. The original survey was made within three months after the bridge was opened to traffic in 1935. The second was made in 1947, and subsequent surveys have been made at regular intervals, the latest having been conducted in October, 1970. A total of approximately 19 measurements at each point have been made since 1947. The control points established consist of bronze plugs set into the top course of granite masonry at the four outside corners of each channel pier, and two points in the concrete at intermediate Piers 3 and 4, a total of 12 points. Concrete monuments were set on shore between the intermediate and channel piers on line with the plugs set in the piers and are used to check for both longitudinal and horizontal motion of the piers. Elevations were read at each of the bronze plugs in order to check for any settlement of the foundations. A review of the survey data taken over the years may be summarized as follows:

Elevations

Six points now higher (maximum 3/16 inch) than in 1935.

Six points now lower (maximum 5/16 inch) than in 1935.

Horizontal Alignment

Measurements indicate gradual shortening of the distance between the concrete monument on shore and the east pedestal of the south channel pier.

The changes in elevations indicated are not of significance considering the accuracy of this type of survey. Slight movements of the bench marks also may have occurred.

The gradual shortening in the horizontal alignment of approximately 5/8 inch would not normally be insignificant in the 48-foot measurement. However, this shortening occurred only on the east pylon measurement, while the dimensions at the west measurement increased slightly. In addition, the concrete monument used initially was destroyed in 1963 and a new monument established. All of the shortening recorded has occurred since 1963, indicating that this monument may not be properly embedded, or that it is too near the canal. The records also indicate that the measurements between the original bound and the pier had been lengthening slightly prior to the destruction of this bound.

During the construction of Piers 1, 2, and 6, certain deviations from the contract drawings were permitted, including the addition of timber piles at Pier 2. These construction changes are shown on the plan titled "Bourne Bridge Foundations As Built" (Plate III). All other foundations were constructed in accordance with the contract drawings.

As part of the investigation of foundations, a plot has been made of the locus of high and low points from the maintenance sections taken between the years 1957 and 1969. These sections, which are a plot of channel soundings taken in the vicinity of the piers, are shown on the twelve drawings listed as data Item 4. Loci of high, low and most recent soundings are shown on Plates IV and V. This provides a graphical history of the limits of change in channel bottom adjacent to the piers during the period of years stated. These data show an average variation of approximately 5 feet in the channel bottom highs and lows with no indication of any severe scouring having taken place.

There is a minimum of 35 feet of soil between the bottom of channel and the tips of the cofferdam sheet piling. This is approximately 10 feet more cover than exists at the Sagamore Bridge.

Our studies of the available survey data, soundings, and records of borings indicate that the foundations of the Bourne Bridge are stable.

SUBSTRUCTURE

The cracks on top of the pylons noted in the following discussion for the various piers should be repaired in the same manner that the 1969 repairs were made. The tops of the piers are relatively accessible and will not require expensive rigging. These repairs are particularly important on the intermediate piers as the pylons at these piers are unreinforced. If the cracks are allowed to weather and widen, large spalls could occur.

During future inspections, the condition of all cracks which have been patched should be noted and the piers and abutments inspected for any new ones which may occur.

Consideration was given to the reason for the occurrence of these cracks in the substructure concrete with the following conclusion. When the concrete originally cured hairline shrinkage cracks formed on the surface which were not perceptible until after many years of weathering. Because of the mass of the concrete in the substructure units their interiors are not subject to daily temperature fluctuations which affect the concrete at and near the surface. This creates a condition which results in the formation of hairline cracks similar to the original shrinkage cracking or to the further opening of existing cracks.

The tie beam sections (arched horizontal members between pylons) are reinforced but support no gravity loads from the structure. However, under lateral loads they insure rigid frame action of the piers. A study of the elevations which were periodically taken on Piers 1 through 4 does not indicate any significant differential settlements between the pylons of any one of these piers. Since most of the cracks in the tie beams are located at points where lateral loads do not cause stress, it is our conclusion that the cause of cracking at the tie beams is the same as described for the pylons.

North Abutment

The general condition of the north abutment was found to be excellent. In 1969, all of the piers and abutments were patched in the areas where cracks were found. These patches appear to be in excellent condition. Due to the relatively short time since the patches were placed, it is recommended that they continue to be checked as a regular maintenance procedure. The following photographs show the north abutment:

Breastwall	-	(A1 to A6)
East Wingwall	-	(A8 to A10)
West Wingwall	-	(A11 to A15)

The east wingwall is in excellent condition with no new cracks or spalls evident. At the end of the west wingwall, the surface patch under the light pole was deteriorating with horizontal and vertical cracking noted. This area is shown in photographs (A14) and (A15). The slopes at this end of the west wall need occasional maintenance to correct the condition shown by photograph (A14).

Inspection was made of the inside of the north abutment. The general condition here is also excellent. Photograph (A7) shows the inside of this abutment. Inside the abutment at the expansion joint that is approximately 43 feet from the end of the abutment, a leak was noted over the second diaphragm wall. Leakage has occurred along the entire length of the wall. The compression seal in the deck joint at this point appears to have deteriorated or to have been improperly installed and should be replaced.

On the west bearing seat of this abutment, a hairline crack was found on the west side of the bearing radiating from the southwest anchor bolt. This crack, shown in photograph (A4), extends down the side face of the abutment approximately 6 inches.

Photograph (A5) shows the concrete backwall under the expansion joint over the bearings. This concrete is in good condition. Photographs (A17) and (A18) show the granite capstones on the wingwall parapets for this abutment. They are in very good condition and in proper alignment. Photographs (A16) and (A19) show the metal sleeves at the end closures for the railings of Span 6. Photograph (A19) is a close-up of a concrete spall at the east wingwall closure. Photograph (A6) shows the condition of the slopes in front of the north abutment. Gravel has been placed to fill a gully, apparently eroded by the water falling from the deck scupper downspouts.

South Abutment

The south abutment is in very good condition. Photographs (A20) through A31) show the general condition of this abutment.

In the concrete breastwall a long crack runs vertically down the center of the wall. From the inside of the abutment, this crack was observed to go from the inside ground line up to midway between the two concrete struts. Evidence of fresh leakage from the outside ground line down was observed. This crack was patched from the outside above the ground line. This patch is shown in photograph (A25) and photograph (A24) shows the condition at the inside wall. No new cracking or spalling was observed in either the east or west wingwall.

There is a large spalled area in the concrete beam under the west curb. This spall is under a roadway scupper and is a result of water and deicing chemicals running down the side of the beam. It is recommended that a splash plate be installed to deflect the drainage from the beam. Photograph (A28) shows this area. Inside the abutment a crack has formed in the northeast corner of the second diaphragm wall from the north. This crack is not structurally significant but is noted for future reference.

Photographs (A30) and (A31) show the south abutment parapets. In photograph (A31) misalignment of the capstone at the east parapet can be seen. The north capstone is 3/8 inch east of the south stone, these stones being the tenth and eleventh stones from the south abutment pylon.

The west side parapet is shown in photograph (A30). The wall joint material in the second joint back from the abutment appears to be overcompressed, as shown in photograph (A29). At this point, the capstones were misaligned, the north stone being 1/4 inch east of the south stone. The distance between the concrete faces under the capstone is 5/8 inch.

The misalignment measurements noted above will be useful in determining if any further movement of the walls takes place. The condition does not warrant immediate correction.

Channel Piers Granite

The two channel piers, Piers 1 and 2, were inspected for the condition of the granite facing at the water line. Inspection was made from a small boat. An effort was made to inspect the granite at low tide so that as many masonry joints as possible could be observed. The granite stones are 2 feet high, and of variable length and depth.

The general condition of the granite stones and the mortar appeared to be very good. The following paragraphs describe areas where the mortar has been eroded, and we recommend that these areas be patched to preserve the integrity of the interior concrete.

Pier No. 1 - Granite

Photograph (A33) shows an opening about 6 inches long in the second mortar joint from the top. This is on the east face of the east pylon. The mortar here chipped off fairly easily and upon close inspection it was found that the inside of this hole was eroded to a considerable extent. A similar condition was found in the third joint down on this face. The rest of this face appeared to be in good condition. On the north face of the east pylon, the mortar surrounding the top corner stone is in poor condition.

The west pylon is in generally good condition. On the north face of this pylon a separation between the mortar and the stone was found at the northeast corner. The second joint down on this face was in poor condition, the mortar being missing in places. The east and west faces were in good condition, with only a hairline separation between the mortar and stone at the northeast corner of the east face. On the south face, cracks were found along the east and west corners near the top. Also noted on the east face of the west pylon, where the pier concrete meets the top granite stone, surface spalling and cracking was noted.

<u>Pier No. 2 - Granite</u>

The granite facing for Pier 2 is generally in good condition. On the west face of the east pylon, at the first joint from the top, the mortar chipped off easily for about 6 feet from the northwest corner, with the first foot from this corner losing mortar up to 2 inches deep. At the fourth joint, the mortar has been eaten away for the first 6 feet from the southeast corner. Also, at the northeast corner of this face, the fourth joint of mortar has deteriorated along a length of 2 feet. Photographs (A37) and (A35) show this condition at the northeast corner of the east pylon. The depth of the deteriorated mortar in this joint varied from 1 to 3 inches. Photograph (A36) is also of the north face of the east pylon, showing the northwest corner stone. The mortar is missing in the vertical joint for depths up to 2 inches with the horizontal joint showing some deterioration also.

Photograph (A38) shows the general condition of the south face of the west pylon. At the southeast corner stone of this pylon the mortar surrounding this stone is in poor condition.

The repair of the mortar joints in the granite facings noted should be done as a preventative maintenance measure. No serious deterioration has yet taken place.

CHANNEL PIERS

The channel piers are gravity structures composed of two hollow shafts tied by a hollow cap beam and supported on a concrete monolith twenty-five feet thick. The shafts above the granite faced pedestals, and the cap beams are reinforced with structural steel frames. This was a substitution made for the bar reinforcement shown on the pier design drawings (Plate VI).

Pier 1

The concrete inspection of Pier 1 indicates the pier to be in very good condition. Photographs (A39 through A45) show the general appearance of the faces of the pier. All of the cracks that were previously patched appeared to be in good condition and have not opened up. The condition of the concrete around the bearings and arch was given special attention.

On the west pylon, small spalls were found on the top surface. All four corners at the top of the pylon have been patched, and a crack running parallel to the east face of the west bearing has also been patched. At the north end of this patch is a continuation of the crack that either was not patched or appeared after the patching was done.

The east pylon has patched areas similar to the west, with an additional crack found on the south face as shown in photograph (A46). This is a hairline crack about 2 feet from the southeast corner, running 8 inches vertically and 8 inches horizontally on the surface of the chamfer.

Pier 2

The condition of Pier 2 is very good. The photographs showing the general condition and features for this pier are numbered A47 through A53. All of the previous cracks that were grouted were examined and found to be in good repair. An additional crack on the south face of the tie beam near the top was noted.

The concrete at the top of the piers was inspected for hairline cracks and new spalling. All four corners of both pylons had previous patches which are in good condition. The west pylon has a hairline crack on the west face chamfer as shown in photograph (A52).

On top of the east pylon, there are three cracks found that were not patched. There is a hairline crack on the north face approximately 3 feet from the northeast corner running vertically down the chamfer for about 1 foot length. On the east face photograph (A53) shows a hairline crack in the middle of this face. Also noted was cracking and superficial spalling at 5 inches from the masonry plate of the east bearing. This crack runs parallel to the west face of the bearing.

INTERMEDIATE PIERS

Pier 3

Pier 3 is also in good condition. Photographs relating to this pier are numbered A54 to A63.

All of the patches that were made in the 1969 repair contract are still in good condition. On the east pylon at the bottom of the west face, some spalling has occurred. On the west pylon there is also spalling near the base of the east face. No repair work is recommended for these areas.

The top of the concrete cap for this pier was inspected, and several cracks and spalls were found. These are shown in photographs (A59 through A63). Photograph (A62) shows a crack going from the east face of the bearings to the edge of the concrete and down the chamfer of the east pylon. Photograph (A56) shows an extension of this same crack going one-third of the way down the east face of the pier which was patched during the 1969 repairs.

Photograph (A63) shows spalling along the north side of the top of the east pylon. There are also two hairline cracks going from the baseplate to the edge of the chamfer at this location. Many of the cracks mentioned for these piers seem to radiate from the anchor bolts for the bearings.

Photograph (A60) shows the north face of the west pylon. Two hairline cracks were found running north-south from baseplate to chamfer. In addition, there is a 4-foot discoloration of the concrete here, and a spall about 12 inches by 6 inches by 3/4 inches. Photograph (A61) shows the other crack on this face. One of these cracks runs 1 foot down the face of the pylon, the other going 3 inches down the face. Also on the west pylon is a crack going north-south at the southeast corner of the south bearing. This crack runs 6 inches down the face of the chamfer, and is shown in photograph (A59).

<u>Pier 4</u>

Pier 4 has the most serious spalled area encountered during the inspection on the piers. On the south face of the east pylon there is a 5-foot long cracked area that is in immediate danger of falling off. This crack, shown in photographs A71, A73, A74 and A75 has discoloration indicating that water is leaking through the crack. On this same pylon cap, spalling and cracking is occurring on the east face also. Photograph A72 shows this area and the 3-foot long crack.

On the west pylon cap, the south face is also cracking and spalling. At the spalled area, the crack is approximately 5 feet long. Upon tapping, this area sounded hollow.

Discoloration due to leakage was also evident. Also, on the west face of this pylon is another hairline crack and a spalled area approximately 6 inches by 6 inches by 1 inch.

The rest of the pier appeared to be in good condition, as were the previous patches. Photographs showing the faces of this pier are numbered A64 through A70.

Pier 5

Pier 5 is in generally good condition. Photographs for this pier are numbered A76 through A81. All of the previous patching appeared to be in good condition. Other small cracks and spalls were found on the sides of the pier, but none of any serious consequence.

On the top of the pier, the west side of the west pylon has a crack running from the bearing seat to the chamfer. This is shown in photograph A81 and shows a 2-1/2-foot spalled area adjacent to the crack. The tie beams of all the piers were in good condition.

The east pylon has a spall on the east face. The north face of this pylon has a hairline crack running north-south down the chamfer and a spall approximately 1 foot - 4 inches by 4 inches by 1/2 inch deep.

Pier 6

The general condition of Pier 6 is shown in the photograph books under photographs numbered A82 through A89. This pier is in very good condition, with only minor cracking and spalling found on the sides of the pier. The patch on the east face of the east pylon is in good condition, and the east pylon and arch are in good condition.

On the west pylon, the south face has two small spall areas with hairline cracks, and photograph A86 shows a 3-foot long crack on the west face.

SUPERSTRUCTURE

The inspection of the superstructure indicates that most of the structural steel and deck concrete are in good condition. The greatest amount of deterioration has taken place in the floorbeams and stringer ends adjacent to roadway joints, in the connections of horizontal bracing members and in the lacings of secondary built up members.

There are several factors contributing to the deterioration of the steel at the vulnerable areas noted. In the case of the steel and concrete adjacent to roadway joints, this deterioration can be attributed to the passing of water and deicing chemicals through these joints to the supporting steel. Measures taken in the past, epoxy painting and adding flash plates, have not proved wholly effective and the deterioration continues. Water leaking through the deck is also reaching these areas due to breakdown of waterproofing, particularly adjacent to roadway joints. The integrity of the waterproofing must be maintained and water must be prevented from reaching the steel through the joints. The roadway joints will be discussed separately under the section on expansion joints.

Photograph B144A shows cracking of the bituminous wearing surface which allows water to enter and then to migrate between the pavement and waterproofing until it finds a break in the waterproofing. The larger of these cracks should be sealed to minimize this migrating water.

The connection plates which are most seriously deteriorated are horizontal. They tend to accumulate debris and moisture. Only frequent maintenance of these plates will prevent continuing damage.

Lacing bars, insides of truss members and certain other areas are subject to surface scaling and subsequent further deterioration due to improper cleaning before painting. A procedure for arresting this type of deterioration is discussed in the section on the condition of the truss members.

Following is a discussion of the conditions found in the structural steel and concrete of the superstructure with recommendations for remedial work. These recommendations are summarized, in the "Summary of Recommended Repairs." A detailed "Summary of Superstructure Inspection Notes" taken from field books are tabulated and included as Appendix II. Plate I provides a key plan for location of the various members in the structure and Plates XV through XXVII give details of construction for the bridge superstructure.

Main Truss Members

The primary structural support for the roadway is provided by two lines of trusses, one at each side of the deck, made up of riveted shapes, plates and lacing bars.

The truss members are in good condition structurally. The general condition of these members and their connections is shown by the following photographs:

Span 1 - B1 thru B6, B10

Span 2 - B7, B8, B11

Span 3 - B13

Span 4 - B15

Span 5 - B14

Span 6 - B9

Span 7 - B12

Photographs B12 thru B15 indicate the poor condition of the paint and the resulting surface rust, especially evident on the truss members of Spans 4, 5, 6 and 7. Photograph B8 showing the inside of member L0'E-U0'E indicates complete failure of the paint system. This member lies below a roadway expansion joint and the extensive damage is no doubt due to deicing chemicals which combine with the drainage runoff during the winter months. Except at such locations as noted, the inside surfaces of the truss members were found to be in approximately the same condition as the outside surfaces. Photographs B7, B9, B10 and B11 show the condition of the insides of truss members.

As indicated in the summary of inspection notes, the primary comments refer to surface rust which is a direct result of improper cleaning before painting and subsequent paint failure. There is evidence of paint having been applied directly over rust scale on many of the truss members. If the integrity of the structure is to be maintained and very costly structural repairs avoided in the future, proper cleaning and painting is mandatory. Although built up members employing lacing bars and stay plates are difficult or nearly impossible to clean by hand, there are blast methods available which, while costly, are still vastly preferable to the alternate future structural repairs. It is recommended that a cleaning contract and painting contract be awarded requiring that all areas where scale is evident beneath paint and all areas where paint has failed be blast cleaned, primed and covered with one coat of finish paint. When this work is satisfactorily completed, one coat of finish paint should be applied to the entire structure. The requirements of the specifications should be more stringent than for a normal painting contract and inspection should be more detailed and rigorous. To insure against minor, but continuing deterioration of the steel on a long term basis, every third or fourth painting job should be on this basis.

There was no evidence of joint distortion or member misalignment noted during the inspection of the truss members.

Member L16E-U15E of Span 1 has very heavy rusting of the lacing bars, with one bar broken as shown in photograph B3. Inside of the connection at L10'E of member L10'E-L11'E, Span 1, has rusted up to 1/4 inch deep.

In Span 2, the lacing on many of the members was found to be in poor condition with the lacing bars at the U9'W end of member U8'W-U9'W having lost up to 50% in section.

In Spans 2, 3, 4 and 5, the condition of the members is generally good, although on many the paint is in poor condition.

At the L0W end of member L1W-L0W, Span 6, heavy rusting at the bottom of the top plate was noted.

In Span 7, member U5W-L6W has heavy rusting on its west face at L6W.

Truss Bracing Connections

The truss bracing connections, although generally in good condition, are the most severely corroded parts of the bridge. The summary of inspection notes lists all the connections with deteriorated rivets or gusset plates. The condition of the plates is listed as either surface rust, moderate rust, or heavy rust. Surface rusting is mainly a painting problem, and heavy rusting is used when 25% or more loss of section was found. The moderate rusting nomenclature is used to describe anything in between. Many of the connections listed have rivets rusted 50% or more. All rivets so described should be replaced. Several upper chord gusset plates in Spans 2 and 3 are very heavily rusted, with holes easily made in them with chipping hammers. Examples of these plates are at U3E-N, U4E-N in Span 3, and U4'E-S in Span 2, and are shown in photographs B25, B27 and B26 respectively. These connections and those in similar condition should be repaired. The summary of recommended repairs lists all such connections requiring repairs. Typical conditions at upper chord connections for the rest of the bridge are shown in the following photographs: B24 at U4'E, B23 at U2'E, B31 at U0'E, and B28 at U4E.

The lower chord connections are generally in better condition than the upper chord, although several in poor condition were found. Connection L4'E in Span 2 and L5E and L6E in Span 3, had holes through them. Connection L4'E, located directly under the heavily rusted connection U4'E, is shown in photograph B18. Photographs B21, B20, B16, and B17 show the condition of lower chord connections with surface to moderate rust. Photograph B19 shows a moderately rusted gusset plate at connection L7'W in Span 2.

The inside of these connections was in about the same condition as the outside. The insides of some of the connections have accumulated debris, particularly at joints L9 and L9'.

In Span 1, the bracing members under the suspended floor are connected to the wind chord. These connections are generally in good condition. On the west end of FB11, north side, 12 rivets were found to be 50% or more corroded. On the east end of this floor beam, 3 of 7 rivets have sustained a 75% loss.

On the truss of Span 1, most of the connections are in very good condition. In Span 2, connection U1'E south side, is heavily rusted, with 11 rivets 50% to 75% gone, and the edge of the gusset plate eaten away. Connection U1E, north side, has heavy rust on both the top and bottom gusset plates.

Spans 4, 5, 6 and 7 were generally in better condition at the truss and truss bracing connections than were Spans 1, 2 and 3. The top gusset plate at connection U8E in Span 5 was heavily rusted and chipped off easily.

As is to be expected, the greatest deterioration at the connections occurred in the horizontal plates; especially those on the upgrade side of the floorbeams. Roadway sand and bird droppings deposited on these plates hold moisture which promotes rusting of the steel. It is essential to clean and maintain the paint on these plates in good condition. This may require that they be painted between regularly scheduled bridge paintings.

Truss Bracing Members

The sectional makeup of the truss bracing members is shown on Plates XVII through XXII. For Spans 1, 2 and 3, the bracing members are made up of four angles connected by lacing bars, as shown in photograph B33. For Spans 4, 5, 6 and 7, the bracing members are generally, except for certain lower chord horizontal bracings, made up of two angles back-to-back, as shown in photograph B38. Two deficiencies were noted in the bracing of Spans 4, 5, 6 and 7. Many of the angles of the upper chord horizontal bracing were bowed vertically, either due to their own weight or due to compressive forces. In some instances, they were resting on the catwalk railing as shown by photograph B41. Secondly, excessive long period vibration was noted in the vertical sway bracing. Both sway brace angles are discontinuous at the center connection which reduces the stiffness to that of the gusset plate. This is not at present a serious condition. In future inspections, the gusset at the connection of the sway bracing should be inspected for signs of fatigue. None was noted during this inspection.

The condition of the bracing members is shown in photographs B32 through B52. Most were found to be in very good condition, especially those above the roadway. The following is a summary of the worst conditions found at these members.

On the superstructure in Span 1, member U11'E-U11'W has rust scale on the inside faces of the bottom angles, as shown in photograph B46. Photograph B47 indicated rusting of lacing bars and B45 shows surface rusting on the inclined flange of member L12'W-L12'E.

The suspended floor bracing of Span 1 is composed of a system of diagonals and longitudinal wind chords forming a horizontal truss extending from joints 10 to 10'. These members were found to be in good condition except at their ends, joints 10 and 10', where leakage through the roadway joints has been a continuing source of corrosion. Photographs B44 and B49 through B52 show this condition. Other trouble spots on the suspended floor bracing include member E13-W14, which is heavily rusted between its bottom angles. At point 15'E there is moderate rusting on the insides of the top and bottom wind chord gusset plates, as shown in photographs B127 and B128. Photograph B127 shows a heavy rust spot where cable hanger connector 15'E-N connects to the upper plate of the wind chord. Access to this spot is difficult. Since this is a critical connection, we recommend this spot to be thoroughly sand blasted, inspected and painted.

Photograph B43 shows typical moderate rust on the wind chord.

In Span 2, leakage through the expansion dam at Pier 4 is causing heavy rusting of the bracing members located directly underneath it. Member L0'E-L0'W has heavy rust on its lacing, and some of the lacings of member L0'E-U0'W are completely gone. Damage to these members is shown in photographs B40, B32, and B33 respectively. Member L7'E-L7'W is heavily rusted with some of the lacing 75-100% deteriorated. Finally, members UO'E-Ul'W, Ul'E-U2'W and U4'E-U5'W all are badly corroded between the bottom angles at their east connections, as shown in photograph B42 taken at U0'E.

In Span 3, the lacing bars on members U0E-L0W, L7E-U7W and L7W-U7E are badly rusted with some bars completely gone.

Photograph B34 shows member U7E-L7W. In Span 4, member L8E-U8W has lost approximately half the cross section of one of the outstanding angle legs as shown by photograph B35.

In Span 5, the upper horizontal bracing is heavily rusted in bay 0-1 and bay 7-8, as shown in photograph B36. In Span 6, member L8E-L6W has heavy rust, and in Span 7, the lower bracing in bay 0-1 also has heavy rust, as shown in photograph B38.

Because this bridge is subjected to high lateral wind forces, the bracing members which resist these forces must be maintained to the same high standards as main truss or floor members. Design wind loadings used for this structure are less than those now recommended for use by the American Association of State Highway Officials. The resulting decrease in the factor of safety under high lateral loads makes it absolutely necessary to maintain the structural integrity of all bracing members.

<u>Sidewalk and Curb Supports</u>

The general condition of the sidewalk and curb supports was found to be very good. This is in sharp contrast to the poor condition of these members on the Sagamore Bridge.

Apparently, the sidewalk waterproofing details proved more effective on this bridge. Access to many of these members was difficult in the flanking spans, but detailed observations were made at the manholes located along the west side of the bridge and by climbing the vertical truss members for visual inspection. The manholes are located adjacent to the expansion dams, where deterioration of the sidewalk and curb supports is most serious. Many sidewalk channel nuts, angles and plates were found to be badly rusted. Two sidewalk channel nuts over Pier 3 were almost all gone. Many others were 50-75% gone. Similar conditions were found throughout the bridge for these members, and the "Summary of Inspection Notes" gives a detailed listing of our findings. The more serious conditions found are described below.

In Span 2, there is a gap between the curbside plate and the top flange of floor beam 0'. This area has heavy rusting over all the surrounding steel caused by leakage through the joint. Photograph B61 shows the curb plate and the top flange of floor beam 0' and photograph B62, shows similar conditions at FB10. Also, in bay 0'-1' of Span 2 is a heavily rusted sidewalk angle. This angle, shown in photograph B56, was found to be twisted and deformed due to rusting. In bay 4'-5' of Span 2, one of the nuts in the plate supporting the curb was loose and

50% rusted away. This is shown in photograph B63, which also shows heavy rust on the surrounding steel. Photograph B58 shows sidewalk channel to clip angle bolts on which the nuts are almost 100% gone, and photograph B60 shows a supporting angle with more than 50% of one leg gone.

All of the nuts listed in the "Summary of Inspection Notes" should be replaced, and the other members listed should be thoroughly sandblasted, inspected and painted. If the joints at the expansion dams are made waterproof, further deterioration will be minimized.

Floor Beams

The condition of the floor beams in general is very good. The areas where significant rusting has occurred are primarily at the expansion joints. The floor beam stiffeners, rivets and the horizontal plates supporting the concrete have all rusted to some degree due to leakage through these joints and the deck adjacent to them. The third stiffener from the west end, the one directly under the sidewalk curb, consistently showed the most extensive damage. (See photograph B74).

Splash plates, shown in photograph B66, were added to the floor beams at the expansion joints to protect the bottom flange. Inspection indicates that these plates are working, but since they are not easily removed, the actual condition of the bottom flanges can not be determined.

One indicator that the splash plates are working is found at the catwalk under the expansion dams between floor beams. The bottom flange of the floor beam under the catwalk grating was found to be very rusty, with many rivets rusted 50% or more. This condition is further aggravated by the continued presence of sand and bird droppings on the grating, speeding up corrosion and hampering inspection. This condition is serious due to the fact that the catwalk is located near the center of the floor beams, where the stress in the flange is maximum. Photograph B69 shows that deterioration of the bottom flange is mainly centered at these locations. We recommend that the catwalk grating under the expansion joints be removed, the bottom flange sandblasted, inspected and painted. After cleaning it will be possible to determine the actual loss of metal and whether or not reinforcing plates should be added. If the expansion joints are made watertight, further loss at these locations should be arrested.

The floor beams at the deflection joints 10 and 10' have been subjected to heavy rusting due to leakage through these joints. The summary of inspection notes enumerates the condition of steel at these locations.

Photographs B83 through B88, show points where heavy rusting has occurred on FB10' and the stringer brackets it supports.

In Span 1, floor beam 13' has rusting at four different locations. On its north side, the east end of the top flange edge has moderate rust, as shown in photograph B80. This is apparently due to leakage through the roadway. On the south side both the east and west ends of the top flange are moderately rusted, as shown in photograph B78, and the east end of the bottom flange is also moderately rusted. This damage, although not structurally significant as yet, points up the need to maintain the deck waterproofing. Also, in Span 1, the west end of floor beam 12, around stringer 1 and the buckle plate, has heavy rust.

In Span 2, the east end of floor beam 0' has moderate rust. Photograph B61 shows that leakage through the curb form plate at stringer 9 is causing heavy rust on all the surrounding steel. There is a space between the curb form plate and the top flange of the floor beam. Photograph B31 shows the damage to stringer 9 as a result of this leakage. Floor beam 0' also has moderate rust between the sidewalk channel and stringer 1, and along its bottom flange. Floor beams 3', 4' and 5' all have moderate to heavy rust at their east connections, and we recommend the following rivets be replaced:

FB 3'	2 rivets East End
FB 4'	12 rivets East End
FB 5'	6 rivets East End

The floor beams in Span 3 were generally found to be in good condition. In Span 4, floor beams 0, 2 and 8 have moderate to heavy rust on them. At the east end of floor beam 0, both the stiffener and the plate supporting the end haunch concrete have very heavy rust, with the plate totally eaten away in places, as shown in photograph B67. Also, on this floor beam, the stiffener at the west end is heavily rusted. On floor beam 2 for this span, 5 rivets on the east end of the bottom flange connecting the vertical sway bracing are rusted 25 to 50%, and should be replaced. These are shown in photograph B79. Floor beam 8 is shown in photograph B65 and B66 which indicate the corrosive effect of water flowing through the joint at the gutter lines at the curb and sidewalk sides respectively. In addition, the rivets under the catwalk grating are heavily rusted. Five of them are 50 to 75% gone and should be replaced.

Floor beams 0 and 8 in Span 5 show moderate to heavy rust. Both of these beams are located at expansion dams. Photographs B70 and B71 show the underside of floor beam 0 and the west end stiffeners respectively. Photograph B72 shows bottom flange rivets on floor beam 8, which are described as 10 rivets 25% gone. Also at floor beam 8, the rivets under the catwalk grating are heavily rusted.

In Span 6, floor beams 0, 3, 6 and 8 also are moderately to heavily rusted. For floor beam 0, the east end stiffener is moderately rusty, and the plate supporting the end haunch concrete has moderate to heavy rust for its full length. The east side of the top flange of floor beam 6 shows heavy rust, and floor beam 8 has heavy rust on the east and west end stiffeners, as shown on photographs B74 and B75.

The floor beams in Span 7 are generally in good condition, but the same floor beams, 0 and 8, again show heavy rust at the expansion joints. Photograph B82 shows floor beam 0. In addition, the rivets under the catwalk grating at floorbeam 8 are moderately rusted.

Stringers

Nine roadway stringers carry the deck between floorbeams and vary in span length from 26' to 44'. They are all 27 WF rolled sections of varying weight with the exception of Span 6 which has 24" rolled stringers.

The stringers are generally in excellent condition. The major exception to this statement is, as in the case of other steel, the ends of stringers adjacent to expansion joints. Photographs B89, B90, B91, B92, B96, B98 and B100 are all of stringers at expansion joints, and photographs B105 through B109 of those adjacent to deflection joints 10 and 10'. Close inspection shows a hole in stringer 1 at floorbeam 3, Span 4 shown in photograph B101. Stringers with up to four pencil size holes were found throughout Spans 4 and 6. These stringers were either under the sidewalk or under the curbside (stringers 1 or 9) and the damage is the result of leakage which occurred prior to 1963 repairs. This condition was arrested when these areas were cleaned and painted, and proper maintenance should preserve these members.

To minimize future deterioration, the expansion joints must be made watertight in order to protect the stringers and other members under them. The following is a span by span summary of the worst conditions found for the stringers. For damage to stringers at 10 and 10' reference is made to the summary of inspection notes because of numerous notations at these locations.

In Span 1, bay 12-13, the top flange of stringer 1 at floorbeam 12 is badly rusted. In bay 9-10, the bottom flange of stringers 1 and 3 were pitted to 1/4" depth for the full length of the stringer. In bay 10-11, photograph B95 shows the accumulation of bird droppings on stringer 3. This is probably due to birds roosting on the cables supported by stringer 3. These cables, apparently no longer in use should be removed from the structure.

In Span 2, stringer 9 at floor beam 5' is moderately rusted on both the top and bottom flange, as shown in photograph B93. At floorbeam 0', stringers 1, 5, 8 and 9 all show moderate to heavy rust, as indicated by photographs B89, B90 and B91. These are all at the expansion joint over Pier 4 and indicate that the roadway waterproofing has ceased to be effective at this location.

In Span 3, stringer 1 at floorbeam 0 is heavily rusted on its top flange and web, as shown in photographs B94 and B96. In Span 4, bay 7-8, stringer 9 is heavily rusted at floor beam 8, as shown in photographs B100 and B98. As stated earlier, most of the deterioration in Span 4 is the result of earlier leakage which has been arrested. In most of the bays, stringers 1 and 9 have lost about 50% of their section for the bottom 4" of their web.

Span 5 was found to be in good condition, but in bay 7-8, stringer 5, the top flange has undergone heavy rusting, for about 6' of its length, starting at floor-beam 8.

In Span 6, bay 7-8, stringer 9 shows moderate rust on the edges of the top and bottom flanges. See photograph B99 for detail. Also, in bay 5-6, stringer 9 is heavily rusted at floor beam 6. Photograph B97 shows this condition. This is typical of the end of stringers as described in the summary of inspection notes for stringers 1 and 9 in Spans 4 and 6. No serious deterioration was found on the stringers in Span 7.

The condition of the stringers as described above and in the summary of inspection notes (Appendix II) indicate that the bulk of damage is to the ends of stringers adjacent to the floorbeams. As the stringers are proportioned for moment at the center of their spans, the loss of material which has taken place at the ends so far is not structurally significant.

The damage to the bottom flanges of stringers 1 and 9 caused by previous leakage through the curbs near the center of spans had not progressed to the point of causing overstress prior to its arrest. However, any further deterioration due to leakage cannot be tolerated at these locations and any leakage detected through the curbs or roadway should be stopped. The top flanges of these stringers are encased in concrete and could not be observed. They were uncovered during the 1963 repairs and were found to be adequate at that time.

Underside of Concrete Deck

Much of the deterioration listed in the summary of underdeck concrete inspection notes (Appendix II) undoubtedly existed prior to the 1963 repairs but was not considered serious enough to require repair at that time. This is still true, and the notations are made primarily for future reference in determining the extent of any further deterioration which might occur. The deck concrete is generally in good condition. Transverse hairline cracking and relatively small spalls were the major conditions noted. At some of the spalls, reinforcing steel was exposed, and some of the previous patches were beginning to break loose. Two areas where consistent deterioration was found were at the haunches at expansion joints and in concrete under the sidewalk around the telephone manholes.

As part of the 1963 repairs, the portions of the deck between stringers 1 and 2, and 8 and 9 were rebuilt, using steel buckle plates for the bottom form. The condition of these plates is excellent, except at the ends adjacent to expansion dams. The following is a span by span summary of condition of the lower surface of the deck concrete.

In Span 1, many spalls in the unreinforced haunches at the floorbeams were found. Photograph B110 shows the condition at the floor beams and photograph B111 shows the spalled off concrete as found on the ground under the bridge. At some floorbeams, the haunch concrete is cracked and will fall within a short time. While this loss of concrete at the haunches is not structurally significant and does not require patching, it is recommended that all pieces which are cracked and have not yet fallen be removed. This will prevent possible damage or injury to ships, vehicles or pedestrians passing beneath the bridge. Many small spalls along the stringers were found with reinforcing steel exposed as shown in photograph B113.

In Bay 14'-15', midbay, between stringers 4 and 5, a two square foot area of cracking and spalling was noted, and the concrete was unsound. This is shown in photograph B119. Repairs should be made at this location.

Also, in Span 1, Bay 10'-11' has spalls along the sidewalk channel for almost its full length as shown in photograph B122.

In Span 2, the typical floorbeam haunch spalls were found with rebars exposed at some locations. In addition, hairline cracks going transverse across the deck were noted. These cracks occur throughout the bridge, with some as close as one to three-foot spacing. Some of the cracks were wider than hairline width, as shown in photograph B116, and in some of them discoloration due to leakage through the deck was visible. Photograph B123, taken from the ground under Span 7, is typical of this condition.

Nothing especially noteworthy, other than typical hairline cracking, was found in Span 3. Photographs B115, B118, and B121 show respectively spalling, cracking and rebars exposed, as found in Span 4.

Span 5 had areas of deck spalling throughout the span. The concrete around the manhole at Pier 3 was heavily spalled, with a 3-foot by 1-foot area completely gone. Photograph B120 shows this area. Around stringer 5 in Bay 7-8 is a large area of discoloration due to leakage of water through and beyond the expansion joint along stringer 5 for about six feet. This may be seen as the darkened area in photograph B124. Deterioration to the top flange of stringer 5 has resulted and the waterproofing should be repaired. Photograph B114 shows the concrete haunch under the expansion dam at Pier 3. This concrete is very badly spalled and has reinforcing steel exposed. This is the result of water and deicing salts flowing through the expansion joint above.

Spans 6 and 7 generally were in good condition.

Cable Hangers

The cable hangers are located at each panel point from 11 to 11' and support the floorbeams for this portion of Span 1. There are two cables at each end of each floorbeam, varying in length from 18 feet to 73 feet, suspended from the lower chord of the arched truss. These cables are composed of six strands of 37 galvanized wires each, with an independent wire rope center of seven strands of seven wires each. The resulting cables are 3-1/4 inches in diameter. Photograph B138 shows a close-up view of a cable hanger.

The cable hangers were inspected from the road deck, and the lower chord of the arched trusses. All of the cables were found to be in excellent condition. Photographs B125 and B126 show the condition of the top cable connector sockets and supporting plates. For the bottom connector sockets, the connection at FB16W has two clamp nuts for the cable hanger guide clamps rusted to 25%. The connection at 15'E is in poor condition. Photographs B127 and B128 show the effects at this connector of water leaking through the bearing plate. This area should be thoroughly sandblasted, inspected and painted to insure that the deterioration does not progress and reach the cable. An attempt was made during the inspection of the Sagamore Bridge to X-ray the top of the connector where the cable enters. However, the lead content of the fill proved too much for a safe radiation source to penetrate. Therefore, what is probably the most critical point on the cable cannot be inspected without removal.

During this inspection, the north cables at points 12E, 13E, 16E, and 14'E did not appear to be as taut as the south cables, and at point 15'W the north cable seemed more taut than the south one. This is not considered structurally significant or unusual and is noted here for future reference.

X-rays of the cables were taken to determine if any internal deterioration had occurred. The following cables were X-rayed:

X-Ray No.	Cable No.	Location
1	12-ES - 0°	At Floorbeam
2	12-ES - 90°	At Floorbeam
3	14-EN - 0°	At Handrail
4	14-EN - 90°	At Handrail
5	16-EN - 0°	At Floorbeam
6	16-EN - 90°	At Floorbeam
7	16-EN - B	At Bottom of Floorbeam
8	14'-ES - 0°	At Floorbeam
9	14'-ES - 90°	At Floorbeam
10	14'-ES - B	At Bottom of Floorbeam
11	12'-EN - 0°	At Floorbeam
12	12'-EN - 90°	At Floorbeam

Two X-rays were taken at 90° to each other at each location. This procedure is desirable when X-raying cables. At points 16E and 14'E, a third X-ray was taken at the bottom of the floorbeam as close to the lower connector as possible.

These X-rays were made by the Arnold Green Testing Laboratories of Natick, Massachusetts. A light and dark negative of each X-ray and a report prepared by a radiologist is included as Appendix III. This report states that no deterioration of the cables was noted in the X-rays.

Catwalks, Lamp Posts and Railings

There are two catwalks on the bridge, one traversing the entire length of the superstructure under the roadway and the other located on the lower chord of Span 1 from panel point 11'W to 16W. These catwalks were generally in excellent condition and are shown in photographs B129, B130, and B131. The underdeck catwalk grating has been replaced since the bridge was built. Heavy rusting was noted at almost all of the floorbeams at the expansion joints, under the catwalk grating, as noted in the section covering floorbeams. These locations, as shown in photographs B133 and B134 need to be thoroughly cleaned and painted. At floorbeam 10', the catwalk handrail was rusted to 50% as shown in photograph B132. At the manhole cage at Pier 4, a section of the top rail is missing. In Bay 10-11, the navigation light attached to the catwalk has one bolt missing. This light should be removed, as it is no longer in use.

Photograph B130 shows the extent of the most serious deterioration in the lower chord catwalk grating and photograph B135 indicates the most serious condition found in the underdeck catwalk supports. Proper cleaning and painting will arrest any further deterioration and no recommendation is made for repairs to catwalks.

All of the lamp posts and cable supported lamp brackets were found to be in good condition, with the exception of their anchor bolts. The following is a summary of conditions found at the lamp posts:

East Side

Pier 5 - NE anchor nut 25% rusted

Pier 3 - NE anchor nut 75% rusted
NW and SW anchor nut 50% rusted

Span 3 - FB5 - NE anchor nut 50% rusted

Span 1 - FB13 - 1 of 6 clamp nuts 50% rusted
FB16 - 2 of 6 clamp nuts 25% rusted
FB13' - Bottom bolt head - 75% rusted
Top 2 clamp nuts - 75% rusted
One in middle - 50% rusted

Span 2 - FB5' - SE and NW anchor nuts - 50% rusted

Pier 4 - NW anchor nut 50% rusted
SE and NE anchor nuts 25% rusted

Pier 6 - NE anchor nut 25% rusted

West Side

Pier 6 - SW, SE, and NW anchor nuts - 25% rusted

Pier 4 - NW nut 75% rusted
SE and SW nuts 25% rusted

Span 2 - FB5' - NW anchor nut 75% rusted

Span 1 - FB16 - 4 clamp nuts 90-100% gone
3 rivets 50% rusted and heavy rust on plates

FB13 - 1 clamp nut 90-100% gone

Span 3 - FB5 - NW anchor nut 90% rusted
SE anchor nut 50% rusted

Pier 3 - NW anchor nut 75% rusted

Pier 5 - SE and NW anchor nuts 75% rusted

Photographs B136, B137, and B138 show some of the anchor nuts and clamp nuts that are badly rusted. Particular attention should be given to the clamp nuts on the lamp bracket at FB16W. On four of the six bolts supporting this bracket, the nuts are rusted 90 - 100% and should be replaced immediately. All anchor bolts listed in the above summary as having 50% or more loss should be replaced.

At the time of inspection, a contract to blast clean and paint the bridge railings was in process. Photographs B139 and B140 show the west railing adjacent to the sidewalk prior to cleaning and painting. As may be seen in the photographs, some of the palings have lost more than

50% of their section. The railings and the palings serve to distribute the load between top and bottom rails should the rail be struck by a vehicle. It is important therefore that not too many consecutive palings be in a weakened condition. It is recommended that a paling with a section loss in excess of 50% be replaced. Inspection notes indicate that there are approximately 45 palings so described. At FB4, Span 6, the vertical gusset attaching the east railing post to the floorbeam has four out of five rivets up to 75% gone. These rivets should be replaced.

At locations where the rails are directly attached to the truss members, the bolts should be replaced.

At Bay 7-8 in Span 7, on the third post on the east railing from FB7, the locknuts on the bolts are backed off and should be tightened. This condition is shown in photograph B141.

Access Ladders

Access ladders are provided at eleven locations throughout the bridge. There are two ladders, one inside and one outside, at each abutment, one at each of the six piers extending from the underdeck catwalk to the top of pier and one providing access from the catwalk on the bottom chord of the west truss in Span 1 to the upper chord at pt. 16. All ladders are vertical with safety rails permanently attached to the ladders to accommodate the slides of the safety belts.

The brackets attaching the interior abutment ladders to the abutment walls show varying degrees of deterioration. At each abutment three of these brackets have lost 50% of their section and should be replaced. Photograph B142 shows the condition of one such bracket at the north abutment.

Photographs B143 and B144 show brackets at Pier 1 and 2 respectively which attach the pier access ladders to the sway bracing. These brackets should be replaced. A bolt at the top of the Pier 6 ladder needs to be tightened. The ladder extending from L16W to U16W shown in photograph B146 is provided with a cage in addition to the safety guide. Some of the straps making up this cage have broken loose and should be repaired. One of the cage verticals at the top of the ladder which is loose is shown in photograph B145.

TRUSS BEARINGS

The truss bearings were inspected to determine their condition and to make certain that they are functioning properly. The bearings were inspected both before and after they were cleaned. Photographs A90 through A165 of Appendix I illustrate the condition at each of the bearings. Measurements of the positions of the bearings at the time of inspection were made and are shown graphically on Plate XXIII. Certain differences from details shown on the Contract Drawings for the bearings at Pier 2 should be noted. Ten 9-inch diameter rollers were substituted for the eight rockers originally specified. Guide lugs attached to anchor bolts through the bed plates were substituted for the 2-1/2 inch anchor bolts specified to be located through the bearing plates. One and three quarter inch extension bars have been added to the south side of the bearing plates of these bearings.

Our inspection found the bearings to be functioning properly and indicated that current maintenance procedures are adequate. The only recommended repair at the bearings is that an attempt be made to straighten the southwest anchor bolt at the east bearing on Pier 3 - Span 3, and that loose anchor bolts be pressure grouted.

Pier 1 Bearings

The bearings at Pier 1 are fixed and all of the movement for the center three spans originates from this point. These bearings are composed of cast steel grillages with bearing plates top and bottom. Both the east and west bearing on this pier are in excellent condition. On each of these bearings, there is a gage bar to measure any vertical movement of the bearings relative to the pier. These gages were installed some years ago, prompted by the curling noted to be occurring at the corners of the top bearing plate. The conclusion drawn from measurements at these gages is that the edges have been forced up by the formation of ice in the voids of the grillage and that the bearings are stable. Sheet metal covers, installed to prevent water from entering the grillage, have been effective and the deformation arrested. The measurement for the east bearing was 2-7/16 inches and for the west bearing 2-1/6 inches, as shown in photograph A97.

All of the cover plates, bearing plates, and rivets on the east bearing are in very good condition, with no noteworthy deficiencies found. This bearing is shown in photographs A90 and A91. Photograph A92 shows the inside of connection L9E, which is in need of cleaning.

The west bearing is in equally good condition with only one exception. The top cover plate for the bearing, shown in photograph A95, has been bent up around the corners and edges, as discussed above. The maximum separation was found to be 15/16 inch. Photographs A94 and A96 show the southeast corner and north face, respectively. Photograph A93 shows the inside of truss connection at this bearing where accumulated debris should be cleaned out.

Pier 2 Bearings

The bearings on Pier 2 are in excellent condition.

The west bearing is shown in the following photographs:

East face - A102

South face - A105

North face - A104

In addition, photograph A101 shows the 1-3/4 inch extension bar added to the south face.

The following photographs show the east bearing:

West face - A98

South Face - A99 and A101

North face - A100

The bearing pin nut on the west face is rusted 10%.

Pier 3 Bearings

Pier 3 supports four roller bearings, two for Span 3, and two for Span 5. The condition of these bearings, with the exception of the anchor bolts, is excellent.

The anchor bolts for the east bearing of Span 3 are in poor condition, being loose, bent, and one broken off. Photographs A107 and A109 show the west face of this bearing, and they show that the south bolt is bent 3-3/4 inches to the south, and that the top 3 inches has broken off. Both the north and south bolts on this face are loose. The east face is shown in photographs A106 and A108. On this face, it is the north bolt that is loose (1/4 inch movement) and bent 3-13/16 inches to the south. As shown in photograph A108, these bolts were bent by being frozen to the slot and pulled back with the bearing. The cover plate for the south face of this bearing was not removed as it is embedded in approximately 4 inches of concrete, as shown in photograph A110.

The west bearing for Span 3 is in very good condition. The east face of this bearing is shown in photographs A110 and A112, and the west face in photograph A111. The north anchor bolt on the west face is bent 2 inches to the north, and the south bolt on the east face has been bent approximately 4 inches to the south. The anchor bolts on the east face are loose, but the ones on the west face are not. Photograph A113 shows the north face of this bearing prior to cleaning and greasing.

Similar conditions were found for the bearings for Span 5. Photographs A114 and A117 show, respectively, the west face of the east bearing and the east face of the west bearing. All four of the anchor bolts shown are bent 2 to 2-1/2 inches to the south. These anchor bolts were still tight and all of the cover plates, rollers, rivets, and connections were in good condition.

Photographs A115, A116, and A118 through A120 shows the conditions at these bearings.

Pier 4 Bearings

All of the bearings on Pier 4 are in very good condition. A few of the anchor bolts are slightly bent, and all of the rollers and plates are in good condition. Movement of the rollers was observed under heavy traffic. Photograph A122 shows some sandblast material had been blown into the bearing, as a result of the sandblasting of the gas main risers which was being done at the time of the inspection. These bearings were cleaned of this material after the inspection. The photographs for Pier 4 are numbered from A121 to A134.

Slight separation of the styrofoam from the pin was noted in the bearings, as shown in photograph A134, indicating that they are rotating at the pins.

Pier 5 Bearings

At Pier 5, the two bearings for Span 5 are fixed, and the two for Span 7 are rollers. The fixed bearings are in very good condition. The west fixed bearing is shown in photographs A137 and A138, and the east bearing in photographs A135 and A136. One of the anchor bolt nuts on the west face of the east bearing is rusted approximately 25%, as shown in photograph A136.

The roller bearings for Span 7 are in good condition. These bearings are shown in photographs A139 through A148.

Photograph A144 shows how grease is pumped into the bearings.

Pier 6 Bearings

All of the bearings on Pier 6 are in good condition. The bearings for Span 4 are fixed, and the ones for Span 6 are rollers. Motion was observed in the latter bearings. The bearings on this pier are shown by the photographs numbered A149 through A157.

North Abutment Bearings

The bearings on both of the abutments are fixed. All of these bearings are in good condition. The west bearing at the north abutment is shown in photograph A160. The condition of the inside of the gusset plates at this bearing is shown in photograph A161.

The east bearing for this abutment is shown in photographs A158 and A159. In addition, photograph A159 shows three of nine rivets rusted between 25% and 50%

South Abutment Bearings

The west bearing for this abutment is shown in photographs A164 and A165. This bearing is in excellent condition, with only surface rust noted.

The east bearing for this abutment is shown in photographs A162 and A163. This bearing is in good condition also; five of nine rivets on the west face were rusted up to 25%.

EVALUATION OF EXPANSION JOINTS

There are eight expansion joints on the deck of the Bourne Bridge, where provision for movement was provided in the design. The locations of these joints and the lengths of spans contributing to expansion at each are as follows:

Location of Joint	Span Length Contributing To Expansion
South Abutment	0 (Fixed bearings deflection only)
Pier 5	240 feet
Pier 3	666 feet
Span 1 at Panel Point 10	0 (At end of suspended span)
Span 1 at Panel Point 10'	0 (At end of suspended span)
Pier 4	1252 feet
Pier 6	208 feet
North Abutment	0 (Fixed bearings deflection only)

The roadway expansion joints on this bridge were rebuilt during the 1963 repairs, and the contract drawings for these repairs (1156E-1-1, 10 sheets) rather than the original design drawings must be referred to for details of the existing joints. Photographs B147 through B162 show the condition of the curb plates and the position of the joints at the time of the inspection. Photographs B163 through B170 show the condition of the roadway plates, and photographs B171 through B177 indicate conditions at the underside of the roadway joints.

Our inspection indicates that the joints are structurally in good condition. All of the joints are functioning in the sense that they allow for the movements of the structures with the exception of those at Panel Points 10 and 10'. The joints at 10 and 10' are frozen due to the rusted condition of the steel below these joints. It is recommended that the surfaces between the stringers and the brackets at these locations be freed, and that the bolts which connect the steel at the ends of the suspended spans be replaced.

There are five main requirements that must be satisfied in the design of modern bridge expansion joints. The order of importance of these requirements varies depending upon the location of the bridge. They are listed as follows, in the order appropriate for the Bourne Bridge:

1. Provide for movements in the structure.
2. Provide watertight roadway surfaces.
3. Have acceptable rideability qualities.

4. Require minimal maintenance.

5. Keep noise to a minimal level.

The existing joints on the Bourne Bridge meet requirements 1, 3, and 5. The problem then resolves to a determination of the importance of the remaining requirements, watertightness and low maintenance, and whether or not the existing joints can be altered to satisfy these requirements.

Our inspection of the bridge clearly indicates that the most significant and continuing deterioration of the structural steel is a result of leakage through the roadway joints. Various methods employed in the past to halt this deterioration have not succeeded, and the only solution appears to be in preventing the water from reaching the supporting steel. The maintenance problems associated with the existing joints includes, as the most significant items, repairs to the structural steel. Repairs to the joints themselves will be minimal. Methods of making the existing joints waterproof or providing means for collecting the water flowing through them and conducting it to a point where it would not damage the structure were investigated. Such systems would include splash plates, troughs, and downspouts. This solution to the problem was discarded because the cost of installation and future maintenance to keep it functioning would not be justified when compared with similar costs for improved types of joints which are now available.

The problem of deterioration on the bridge structures due to leakage through expansion joints is not peculiar to the Bourne Bridge. This problem has concerned bridge designers for many years, particularly because of increased use of deicing chemicals on highways.

During the past decade, joints have been developed which adequately meet the five requirements set forth. These joints have been developed through the cooperation of bridge designers and manufacturers throughout the world. Two basic concepts of joints have evolved. The modular preformed seals are composed of a number (depending on amount of motion to be accommodated) of fairly narrow compressible neoprene seals which are precompressed between steel or aluminum separators. Steel bars spanning the joint width supports the steel or aluminum separators. Joints accommodating limited motion are made with only a single seal, preferably mounted between steel angles. The second type of joint consists of a neoprene traffic plate (transflex) in which steel plates are embedded to span the opening. The modular joints are prefabricated and must be installed properly to insure watertightness. The transflex types are usually installed in four to six-foot wide sections which are jointed with an adhesive sealant at their ends. One advantage of the transflex type joints is that it can, in the case of smaller openings, be installed on existing structures directly over existing joint hardware. We have noted this possibility on the sketches of such joints. There is of course some loss of ride-ability if this method of installation is chosen. We have studied the two types of joints outlined above for installation at each location on the Bourne Bridge. Sketches were prepared for each type showing the road-way section and the curbs and sidewalks. The outlines of existing concrete and joint hardware are shown out of function in the sketches. These sketches are shown on Plates XXV, XXVI, XXVII of this report. A comparison was made of the effectiveness of each type in providing the desirable joint qualities and com-parative cost estimates is included. Because the Bourne Bridge is a major traffic link, consideration was also given to the amount of interference with traffic which would result from the installation of each type. A tabulation of comparisons together with our opinions on the relative merits of each type is included in Table I. This tabulation was used as the basis for our recommendations to follow.

After careful study of the details and methods of installation for each of the new types of joints and an evaluation of the existing joints, we recommend that all joints on the Bourne Bridge be rebuilt with the type recommended in Table I.

The cost of the joint was not considered to be the deciding factor in the selection of type. The estimated cost of the combination of joints recommended is $154,000. The cost of engineering design, supervision, and contract administration are not included in the estimate.

The types recommended have been installed in numerous new and existing bridges throughout the country and have proven superior to other types. The simplicity of detail and minimal interference with traffic during installation offered by the transflex joint dictates the selection of this type for larger joints. For the smaller joints at the abutments and at Panel Points 10 and 10', we have selected the preformed seal type.

The sketches showing the joints were prepared to permit comparison and are not sufficiently detailed to serve as construction drawings. Such details as reinforcing steel, anchors, hold down, and centering devices have not been shown, as they would be common to any joint detailed.

When the contract drawings and specifications are prepared for rebuilding these joints, we suggest that particular attention be given to the following items.

The roadway waterproofing detail must be carefully worked out for the types of joints selected so that leakage through the deck behind the joints will not occur.

Consideration should be given to any developments that may occur in joint technology between now and when the construction drawings are prepared. Many improvements, especially for the modular type joints, are currently being developed both in Europe and domestically. These developments are periodically presented before the AASHO Committee on Bridges and Structures.

Cleaning and painting of the structural steel below the joints should be done when they are rebuilt.

TABLE I

COMPARISON OF JOINT TYPES

	At Abutments Section AA		At Piers 5 & 6 Section BB		At Pier 3 Section CC		At Pier 4 Section DD		At Panel Points 10 & 10' Section EE	
	Preformed Seals	Trans-flex	Modular Seals	Trans-flex	Modular Seals	Trans-flex	Modular Seals	Trans-flex	Preformed Seals	Trans-flex
Minimum Interference with Traffic During Installation		X*		X		X		X		X*
Better Rideability	X		X			X		X	X	
Lower Maintenance Cost	X			X		X		X	X	
Lower Noise Level	X		X			X		X	X	
Material Cost Per Joint	$1,100	$ 3,100	$ 7,500	$ 8,000	$28,500	$24,500	$42,000	$43,300	$1,200	$2,800
Installation Cost Per Joint	$5,900	$ 8,900	$ 9,000	$ 9,500	$10,500	$12,000	$10,500	$12,200	$5,300	$6,700
Total Cost Per Joint	$7,000	$12,000	$16,500	$17,500	$39,000	$36,500	$52,500	$55,500	$6,500	$9,500
Joint Recommended	X			X		X		X	X	

*If overlay installation method is selected.

Note: Both type of joints are equal in their ability to provide for the anticipated movements in the structure and in providing a watertight roadway surface.

SUMMARY OF RECOMMENDED REPAIRS

Following is a summation of repairs recommended in this report, the priority of each, and an estimated cost for making the repair. The priority rating system used is as follows:

1. High priority to insure safety and integrity of structure.
2. Repairs to correct deterioration that is not currently significant and to reduce probability of future deterioration and maintenance.
3. Repairs to minor items, improvements in appearance of structure and routine maintenance procedures.

The estimated costs of repairs is based on the assumption that all work in any one category is executed at the same time.

* 1971 PRICE LEVEL USED IN ESTIMATING COST OF REPAIRS

LOCATION	REPAIRS RECOMMENDED	PRIORITY	COMMENTS	COST ESTIMATE*	REPORT PAGE REFERENCE
SUPERSTRUCTURE	GENERAL CLEANING AND PAINTING OF STEEL	2	SEE DISCUSSION OF MAIN TRUSS MEMBERS FOR RECOMMENDATION.	$250,000	38
MAIN TRUSS MEMBERS	SPAN 1				
	U15E-L16E - REPAIR DETERIORATED LACING	1		1,500	40
	SPAN 2				
	U8'W-U9'W - REPAIR DETERIORATED LACING	2		1,500	40
BRACING CONNECTIONS	SPAN 2				
	L0'W - REPLACE 3 RIVETS ON ANGLE TO WIND BRACE	2		130	11 - 4
	U0'E - REPLACE 2 BOTTOM GUSSET RIVETS	2		120	41
	U1'E-S-REPLACE 5 TOP GUSSET RIVETS	2		150	42
	U1'E-S-REPLACE 11 BOTTOM GUSSET RIVETS	1		210	42
	U2'E-N-REPLACE 2 GUSSET RIVETS	2		120	41
	U2'E-S-REPLACE 5 BOTTOM GUSSET RIVETS	1		150	41
	U2'E-S-REPLACE 3 TOP GUSSET RIVETS	2		130	41
	L3'E-REPLACE 3 TOP GUSSET RIVETS	2		130	11 - 4
	U3'E-S-REPLACE 7 TOP GUSSET RIVETS	1		170	11 - 4
	L4'E-REPAIR TOP GUSSET PLATE	1		1,000	42
	U4'E-S-REPAIR BOTTOM GUSSET PLATE	1		1,000	41
	U4'E-S-REPLACE 13 TOP GUSSET RIVETS	1		230	41
	U4'E-S-REPLACE 5 SIDE GUSSET RIVETS	1		150	41
	U4'E-S-REPLACE 4 BOTTOM GUSSET RIVETS	1		INCLUDED IN PLATE REPAIR	41
	U4'E-N-REPLACE 4 TOP GUSSET RIVETS	2		140	41
	L7'W-REPLACE 2 TOP GUSSET RIVETS	2		120	42
	L8'W-REPLACE 2 TOP GUSSET RIVETS	2		120	11 - 3
	SPAN 3				
	U0E-N-REPLACE 17 TOP GUSSET RIVETS	2		270	11 - 5
	U0E-N-REPLACE 17 BOTTOM GUSSET RIVETS	2		270	11 - 5
	U1E-N-REPLACE 1 BOTTOM GUSSET RIVET	2		110	42
	U1E-N-REPLACE 2 TOP GUSSET RIVETS	2		120	42
	L2E-REPLACE 2 TOP GUSSET RIVETS	2		120	11 - 6
	L2E-REPLACE 2 BOTTOM GUSSET RIVETS	2		120	11 - 6
	U2E-N-REPLACE 5 BOTTOM GUSSET RIVETS	1		150	11 - 6
	U2E-N-REPLACE 8 TOP GUSSET RIVETS	1		180	11 - 6
	U3E-S-REPLACE 3 BOTTOM GUSSET RIVETS	2		130	11 - 6
	U3E-N-REPLACE 10 TOP GUSSET RIVETS	1		200	41
	U3E-N-REPAIR BOTTOM GUSSET PLATE	1		1,000	41
	U4E-N-REPAIR BOTTOM GUSSET PLATE	1		1,000	41
	U4E-N-REPLACE 15 TOP GUSSET RIVETS	1		250	41
	U4E-N-REPLACE 5 BOTTOM GUSSET RIVETS	1		INCLUDED IN PLATE REPAIR	41
	L5E-REPAIR TOP GUSSET PLATE	1		1,000	42
	L6E-REPAIR TOP GUSSET PLATE	1		1,000	42
	U6E-N-REPLACE 9 GUSSET RIVETS	1		190	11 - 7

LOCATION	REPAIRS RECOMMENDED	PRIORITY	COMMENTS	COST ESTIMATE*	REPORT PAGE REFERENCE
BRACING CONNECTIONS	SPAN 7				
	L3E-L3W - REPLACE 9 MIDSPAN RIVETS	2		$190	11 - 8
	L4E - REPLACE 5 BOTTOM GUSSET RIVETS	2		150	11 - 8
	SUSPENDED FLOOR				
	FB11E-N - REPLACE 3 GUSSET RIVETS	1		130	42
	FB11W-N - REPLACE 6 BOTTOM GUSSET RIVETS	1		160	42
	FB11W-N - REPLACE 6 TOP GUSSET RIVETS	1		160	42
	FB12E-N - REPLACE 2 BOTTOM GUSSET RIVETS	2		120	11 - 9
	FB16E-N - REPLACE 1 BOTTOM GUSSET RIVET	2		110	11 - 9
BRACING MEMBERS	SPAN 1				
	U9'W-U10'E - REPLACE DETERIORATED RIVETS AND LACING	2		1,500	11 - 10
	BRACING UNDER FB10 AND FB10' - REPLACE RIVETS AND LACING	2	EXTENT OF REPAIR TO BE DETERMINED AFTER BLAST CLEANING	2,000	44
	ENDS OF SUSPENDED SPAN - REPLACE MISSING BOLTS IN SLOTTED HOLES AT WIND CHORD	2		200	44
	FB15'E - SANDBLAST, INSPECT AND PAINT PLATES IN VICINITY OF CABLE HANGER CONNECTOR	1		200	45
SIDEWALK AND CURB SUPPORTS	REPLACE ALL SW CHANNEL NUTS LISTED IN THE SUMMARY OF INSPECTION NOTES	1	APPROXIMATELY 15 TO BE REPLACED	500	48
	SPAN 2				
	FB0'E - SEAL GAP BETWEEN FB0' AND CURBSIDE PLATE TO STOP LEAKAGE	2		200	47
	BAY 0'-1' - REPAIR HEAVILY RUSTED SIDEWALK ANGLE	2		200	47
FLOORBEAMS	AT CATWALKS UNDER EXPANSION DAMS - REMOVE GRATING AND SANDBLAST, INSPECT, REPAIR AND PAINT FLANGES	2	APPROXIMATELY 50 TO BE REPLACED	2,000	49
	REPLACE IN SPAN 2 2 RIVETS EAST END FB3' 12 RIVETS EAST END FB4' 6 RIVETS EAST END FB5'	2		600	51
	REPAIR FB STIFFENERS AT STRINGERS 1 AND 9 INSIDE EXPANSION JOINTS	1	APPROXIMATELY 20 TO BE REPAIRED	2,500	51
	SPAN 4				
	REPLACE - 5 RIVETS EAST END FB2	2		150	51
STRINGERS	STRINGER 3 - SPAN 1 -				
	REMOVE CABLES ATTACHED TO STRINGER	2		200	54
	REPLACE BOLTS IN SLOTTED HOLES AT BRACKETS FB10 AND FB10' AND FREE ENDS OF STRINGERS	2		3,000	53
UNDERDECK CONCRETE	SPAN 1				
	BAY 14'-15' - PATCH 2 SQ.FT. AREA BETWEEN ST4 AND 5	1		200	57
	SPAN 5				
	BAY 7-8 - STRINGER 5 - REPAIR WATERPROOFING NEAR FB8	1		500	58

LOCATION	REPAIRS RECOMMENDED	PRIORITY	Nuts COMMENTS	COST ESTIMATE*	REPORT PAGE REFERENCE
LAMP SUPPORTS	REPLACE ALL DETERIORATED ANCHOR NUTS	1	BOLTS AT FB16W SHOULD BE REPLACED IMMEDIATELY	$1.500	63
ACCESS LADDERS	REPLACE DETERIORATED BRACKETS INSIDE ABUTMENTS AND OVER PIERS 1 AND 2	2	APPROXIMATELY 10 BRACKETS	500	65
	PIER 6 - TIGHTEN BOLT AT TOP OF LADDER	2		30	65
	L16W-U16W - REPAIR CAGE STRAPS	2		100	65
RAILINGS	REPLACE PALINGS WITH MORE THAN 50% SECTION LOSS	2	APPROXIMATELY 45 PALINGS	1.000	64
	REPLACE 4 RIVETS AT RAILING POST AT FB4E, SPAN 6	2		100	64
	REPLACE ALL BOLTS WHERE RAILING IS ATTACHED TO TRUSS	2		500	64
	SPAN 7E, THIRD POST FROM FB7, TIGHTEN LOCKNUTS	2		20	64
TRUSS BEARINGS	PRESSURE GROUT LOOSE ANCHOR BOLTS	2		500	66
	PIER 3, EAST BEARING FOR SPAN 3, STRAIGHTEN SW ANCHOR BOLT	2		200	66
	PIER 1 - CLEAN OUT INSIDE OF GUSSET PLATES	2		100	68
SUBSTRUCTURE PIER CAPS AND BEARING SEATS	GROUT CRACKS ON TOP OF PYLONS	1	REPAIR SIMILAR TO 1969 REPAIRS	1,500	22
NORTH ABUTMENT	REPLACE COMPRESSION SEAL IN DECK JOINT	2		500	24
SOUTH ABUTMENT	INSTALL SPLASH PLATE UNDER WEST CURB ROADWAY SCUPPER	2		300	26
CHANNEL PIERS GRANITE	PATCH ERODED MORTAR	2		750	27
PIER 4	PATCH CRACKED AREAS ON SOUTH FACE OF EAST AND WEST PYLONS	1	EAST PYLON PATCH IS IN IMMEDIATE DANGER OF FALLING OFF	500	33
EXPANSION JOINTS	REPLACE ALL JOINTS WITH WATERTIGHT SEALS	1		$154,000	
			TOTAL	$440,250	
			CONTINGENCIES	$65,750	
			TOTAL ESTIMATED CONSTRUCTION COST OF REPAIRS	$506,000	
ADDITIONAL PERIODIC INSPECTIONS AND MAINTENANCE RECOMMENDED	SUBSTRUCTURE				
	PERIODICALLY INSPECT CONDITION OF CRACKS IN PIERS AND ABUTMENTS. PERIODICALLY INSPECT CONDITION OF PARAPETS AND WINGWALLS FOR ALIGNMENT	3	Pat Storey – 15 Aug 72 All nuts to be replaced on west side. East side to be done later		22
	SUPERSTRUCTURE				
	MAINTAIN INTEGRITY OF DECK WATERPROOFING OVER ENTIRE LENGTH OF BRIDGE, INCLUDING SEALING OF CRACKS IN PAVEMENT TO MINIMIZE MIGRATING WATER. FOR STRINGERS IN SPANS 4 AND 6, NO FURTHER LEAKAGE CAN BE ALLOWED TO STRINGERS 1 AND 9.	1			36
	FREQUENT CLEANING MAINTENANCE OF THE PAINT SYSTEM ON HORIZONTAL CONNECTION PLATES	3			37
	PERIODIC INSPECTIONS OF VERTICAL SWAY BRACING FOR SIGNS OF FATIGUE IN CENTER CONNECTION	3			44
	PERIODICALLY INSPECT FLOORBEAM HAUNCHES UNDER DECK FOR SPALLED CONCRETE AND REMOVE LOOSE PIECES	3			57

BOURNE BRIDGE
1971 CONDITION REPORT
SUMMARY OF RECOMMENDED REPAIRS

COMPARISON OF CONDITION VS. SAGAMORE BRIDGE

Because the center three spans of the Bourne Bridge are identical to the Sagamore Bridge and were constructed at the same time, it was felt that a comparison of the condition of the two bridges would be helpful. The comparison indicates that the structures have suffered the same types of deterioration to approximately the same general degree due to the same causative factors.

Our inspection of both structures indicates the major concern is the leakage of water through the roadway joints. Both structures have had major renovation work done within the last ten years. The following are specific comparisons made regarding the two structures.

The roadway joints on the Bourne Bridge were repaired as part of its 1963 renovation and as a result, are structurally in better condition than those on the Sagamore Bridge. It should also be noted in this regard that the open saw tooth type joints, which replaced the joints at panel points 10 and 10' in Span 1 of the Bourne Bridge, allow quicker drying of the water which falls through the steel beneath, than the closed sliding detail remaining on the Sagamore Bridge.

The supporting steel under the sidewalk and curb at the Bourne Bridge is in significantly better condition than that at the Sagamore Bridge. This is because a superior waterproofing detail was used on the Bourne Bridge at the junction of the granite curb and the sidewalk. In 1969, the sidewalk and curb of the Sagamore Bridge were waterproofed and further deterioration of the sidewalk supports arrested. The paint on the Sagamore Bridge appeared to be in better condition than that on the Bourne Bridge. This observation allows for the difference in time since the last painting of each structure and applies primarily to the presence of scale under the surface paint.

The underdeck concrete, truss bracing, and truss bracing connections appear to be in better condition at the Sagamore Bridge.

The substructure of both bridges is in comparative condition, both having had surface cracks grouted as part of the 1969 structural repair contract. As a detailed inspection of the Sagamore Bridge substructure was not included in our 1969 inspection, it was not noted whether the cracking at the top of the pier pylons found at the Bourne Bridge also exists at the Sagamore Bridge. It is recommended that the top of the Sagamore Bridge piers be inspected for this cracking and if found, they be repaired as recommended for the Bourne Bridge.

A comparison of the vibration measurements made for the Sagamore Bridge and those made for Spans 1, 2, and 3, of the Bourne Bridge indicate similar vibration characteristics at the frequencies studied.

PROFILE SURVEY

During the course of the investigation a profile survey was run along the top of the granite curbing on the west side of the bridge. Elevations were read at each panel point and the dimension from the top of curb to roadway surface was recorded. From this information a theoretical profile at the centerline of roadway was computed from the curb profile and the known cross slope of the deck. The necessity of interrupting traffic was eliminated by this procedure, and the resulting profile will make a more permanent reference profile for future inspections since the roadway is repaved periodically. These profiles and the original design profile are shown on Plate XXIV.

The present theoretical centerline profile is a maximum of 3.5" higher than the original design profile at the center of Span 1, and about 3" lower at Piers 3 and 4. Of the remaining areas of the roadway, the theoretical profile was within approximately 1" of the original design profile. The differences noted can be attributed to changes in pavement thickness, temperature, and jacking of the bearings during construction required by construction procedure. No records of an as-built profile were found, and the differences are within the expected tolerances for this type of construction.

Other survey data taken during the inspection which may be useful for future reference is listed below:

CABLE OFFSETS

West Side		East Side	
Point	Offset of Bottom of Member from Top, in Feet	Point	Offset of Bottom of Member from Top, in Feet
11S	S .04	11S	S .040
11N	S .04	11N	S .030
12S	S .04	12S	S .035
12N	S .035	12N	S .040
13S	S .030	13S	S .035
13N	S .025	13N	S .035
14S	S .030	14S	S .010
14N	S .020	14N	.00
15S	S .025	15S	N .005
15N	S .025	15N	N .005
16S	S .045	16S	N .020
16N	S .050	16N	N .015
15'S	N .010	15'S	N .030
15'N	S .015	15'N	N .040
14'S	S .030	14'S	N .040
14'N	S .030	14'N	N .040
13'S	S .020	13'S	N .040
13'N	S .020	13'N	N .035
12'S	.00	12'S	N .050
12'N	S .005	12'N	N .045
11'S	N .015	11'S	N .040
11'N	N .015	11'N	N .040

DEFLECTIONS AT MIDDLE POINTS OF SPANS

Deflections in Feet

Span No.	No Traffic	Light	Medium	Heavy
1	0	0.015	0.03	0.08*
2	0	0.01	0.03	--
3	0	0.01	0.03	--
4	0	0.01	0.03	0.03
5	0	0.01	0.01	--
6	0	0.00	0.02	0.03
7	0	0.00	0.015	--

*Heavily loaded trailer

TRUSS VERTICALS

Span and Point	Offset of Top of Member from Bottom, in Feet	Span and Point	Offset of Top of Member from Bottom, in Feet
7-E0	N .02	7-W0	N .04
7-E8	S .02	7-W8	N .01
5-E0	N .015	5-W0	N .045
5-E8	S .06	5-W8	S .06
3-E0	N .07	3-W0	N .10
3-E9	N .011	3-W9	N .16
2-E9'	S .19	2-W9'	S .16
2-E0'	S .065	2-W0'	S .075
4-E0	N .065	4-W0	N .09
4-E8	S .01	4-W8	.00
6-E0	N .04	6-W0	N .045
6-E8	S .005	6-W8	.00

TRAFFIC DATA AND LOADING ANALYSIS

Introduction

Available traffic survey data has been analyzed and evaluated to establish a basis for comparing actual, or predicted, live loads with the design live loads that are part of the design criteria for the project.

Traffic surveys have been performed periodically by the Massachusetts Department of Public Works, Traffic Planning Section. Two-way, hourly, unclassified vehicle counts were conducted at Station 707, the Bourne Bridge, during a minimum of seven days of each month during 1969 and 1970. Summaries of the data obtained in this survey define the hourly, daily, and monthly variations in traffic volumes at this station. Several characteristics of bridge traffic are revealed by these data and summaries. Summer season volumes greatly exceed those of other seasons. The peak 24-hour summer traffic volume is approximately 1.8 times the annual average daily traffic volume. Saturdays exhibited the highest volumes, followed by Fridays and Wednesdays as shown in the following table.

TABLE 1
BOURNE BRIDGE
24-Hour Traffic Volumes
Two-Way Totals

Day of Week (by order of magnitude)

	Saturday	Friday	Wednesday
Annual Average Volume	18,500 vpd	16,750 vpd	13,750 vpd
Summer Average Volume*	31,200	26,400	21,600
Peak Volume	33,350 (in July)	30,620 (in July)	25,370 (in August)

*Summer was defined as June through August

Source: Massachusetts Department of Public Works, 1969 and 1970 Traffic Counts

Vehicle Classification Distribution

No vehicle classification counts were available for traffic crossing the Bourne Bridge. A reasonable estimate of vehicle-type distribution may be obtained by analyzing the classification counts taken at the Sagamore Bridge, a facility with similar traffic characteristics. This analysis leads to the following percentages of each type of vehicle which may be expected in a typical queue.

Unclassified (Passenger, Light Commercial)	84%
Single Unit Trucks (Two and three axle)	11%
Tractor Semi-Trailer Units (Three to five axles)	4%
Buses	1%

Peak Loading

The objective of correlating vehicle classification information with vehicle counts is to establish a probable maximum truck density in a traffic stream stalled on the bridge. This is not a conventional measure. Therefore, a procedure was formulated. The following general assumptions are based on analysis of collected data and information, observation of specific site characteristics, and application of common knowledge and traffic engineering principles.

1. The maximum live loading must occur when traffic is halted at a time when truck density is maximum.

2. The maximum truck density will occur during a non-summer month. March is considered representative.

3. Maximum truck density is represented conservatively by applying average maximum hourly truck counts to average corresponding hourly vehicle count at the bridge during selected weekdays in March.

4. In the absence of specific information on directional splits, the use of an even split is conservative in the determination of truck density. Similarly, applying the same truck values to each direction of travel on the bridge is conservative.

Peak Loading Analysis

In order to determine the effect of a peak load (traffic halted) on the structure, we have assumed the vehicle distribution above and computed the equivalent uniform lane load on the following basis:

Type Vehicle	Average Weight Lbs.	Lane Length Occupied (feet)	% of Total Volume
Passenger and Light Commercial	4,000	25	84
Single Unit Trucks and Buses			
One-half	30,000	35	12
One-half	40,000	35	
Tractor - Semi-Trailer Units			
One-half	54,000	50	
One-half	72,000	50	4

Calculation of Equivalent Uniform Lane Load

Assume 2000 feet of loaded lane, x = No. Vehicles in 2000' of lane

.84 x (25') + .12 x (35') + .04 x (50') = 2000' (Assumed)
x = 74 vehicles in 2000' of lane

.84(74) =	62	Passenger Vehicles @ 4,000#		= 248,000#
.12(74) =	9	Single unit trucks & Busses @	35,000#	= 315,000#
.04(74) =	3	Tractor-Semi-Trailer Units @	63,000#	= 189,000#
	74			752,000#

Equivalent Uniform Lane Load = $\frac{752{,}000\#}{2{,}000'}$ = 376#/lin. ft. in one lane

Because a continuous queue of traffic was assumed in computing the equivalent uniform lane load of 376 pounds per linear foot, only a more unfavorable vehicle type distribution could cause an increased load. With the original structure design value of 525 pounds per linear foot per lane (plus concentrated loads), the structure can support in excess of 140 per cent of the computed load without exceeding design stresses. Therefore, no foreseeable change in traffic pattern should cause an overstress in the main truss members which are proportioned for lane loading.

LOADING AND STRESS REVIEW

Loading - Dead Load

The renovation of the deck, carried out in 1963, resulted in a slight increase of dead load on the structure. The Haydite concrete was removed between stringers 1 and 2 and between stringers 8 and 9 along the length of the bridge. Steel buckle plates were added between these stringers. The new lightweight concrete deck between these stringers has approximately 10% greater average thickness than that of the original structure. In addition, granite curbs were added at the sidewalk and safety curbs. These changes have been shown on plate XV.

Loading - Live Load

The Bourne Bridge was designed substantially in accordance with the H20 live loading of the "Standard Specifications for Highway Bridges and Incidental Structures" adopted by the American Association of State Highway Officials in 1931. The current design criteria for live loading for this structure would be the HS20 loading, described in the "Standard Specifications for Highway Bridges" adopted by the American Association of State Highway Officials, Tenth Edition, 1969. A summary of the loadings is given on the following pages.

1. <u>Lane Loading (for design of main truss members)</u>

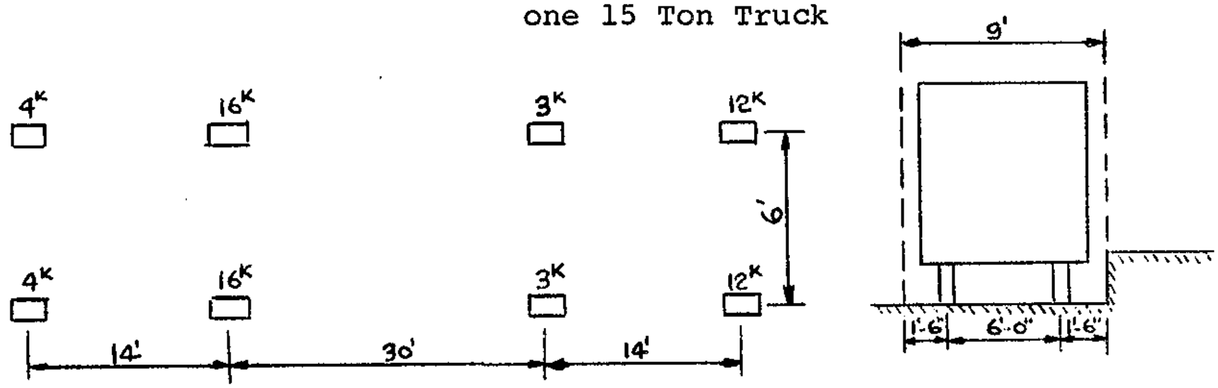
The difference in the specified lane loading between the 1931 and 1969 specifications is that the width of the design lane is nine feet in the 1931 specification and ten feet in the 1969 specification. The Specifications also have different reduction factors for simultaneous loading of more than two lanes. The lane loading, modified by the applicable reduction factor for four lanes loaded, are presented graphically below -

3/4 x 640 -

1931 AASHO		1969 AASHO
(14,800# for Moment (21,400# for Shear	Concentrated Loads	(13,500# for Moment (19,500# for Shear
525#/LF		480#/LF
(9' Lane Width) 1931 AASHO		(10' Lane Width) 1969 AASHO

2. <u>Truck Loading (for design of Stringers, Floorbeams, Cables, and Secondary Truss Members)</u>

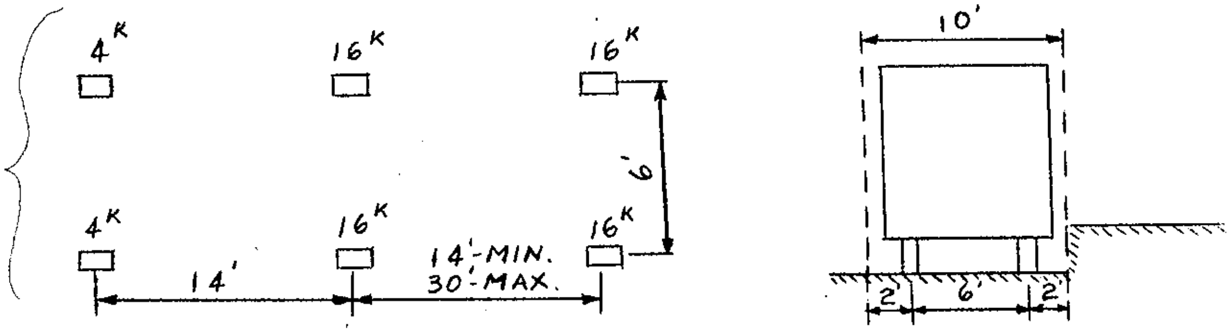
1931 Specification - One 20 Ton Truck Followed by one 15 Ton Truck



Distribution Factor for Stringer Design
Fascia Stringers = Simple Beam Distribution
Interior Stringers = S/4.5 S = Stringer spacing

1969 Specification - One 36 Ton Tractor and Semi-Trailer Combination

HS 20



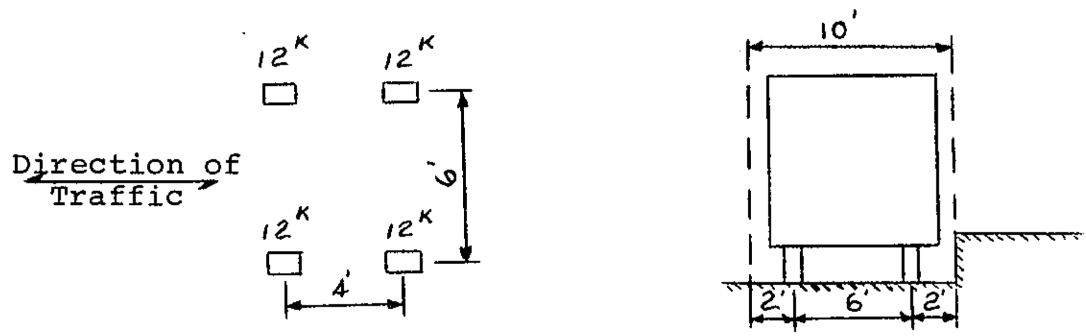
Distribution Factor for Stringer Design
D.F = S/5.5   S = Stringer spacing

Alternate load for 1969 Specification

Military load used in the Design

of Interstate Highways

(Governs Design of Stringers
with Spans between 11 feet and 37 feet.)



3. Impact Loading (As a percent increase in Live Load)

1931 $I = \frac{50}{L+125}$

1965 $I = \frac{50}{L+125}$ Maximum of 30 per cent

L = Length of span loaded to produce maximum stress

4. Sidewalk Live Loading

1931 - 100#/SF For design of immediate supports
45#/SF For design of truss members

1969 - 85#/SF For design of immediate supports
30#/SF For design of truss members

Loading - Wind Load

1931	Wind on Structure	-	45#/SF	On area seen in Elev.
	Wind on Live Load	-	200#/LF	
1969	Wind on Trusses	-	75#/SF	
	Wind on Girders and Beams	-	50#/SF	
	Wind on Live Load	-	100#/LF	

Stress Review

1. Allowable Stresses

The following modifications to the allowable stresses given in the 1931 AASHO are included in the original contract specifications.

> "Paragraph 305, Unit Stresses - Unit Stresses for Carbon Steel are those specified 'for dead load', Section 4 of Division V of the Standard Specifications, with a further limitation that the unit stresses resulting from the application of the live loads, combined with dead loads and impact, are in no case in excess of three-fourths of the allowable unit stress 'for dead load'.
>
> "For lateral forces the allowable unit stresses given for 'live load and lateral forces' are increased by twelve and one-half per cent.
>
> "For Silicon Steel, unit stresses are increased 35 per cent over those for Carbon Steel."

A copy of the allowable stresses given in the 1931 AASHO specification, expanded to include the above mentioned modifications, is included in Table 1 on the following page.

TABLE I

C SIL. C S

INV OP

18 24 24.5 33.5

	ALLOWABLE STRESSES 1931 AASHO 5.4.2. Structural Grade and Rivet Steel (1931 Edition)		ALLOWABLE STRESSES AS MODIFIED BY CONTRACT SPECIFICATIONS	
			CARBON STEEL	SILICON STEEL
	For live load and lateral forces	For dead load	For dead+live+ impact and for lateral forces	For dead+live+ impact and for lateral forces
Tension:				
Axial tension, structural members, net section......	16,000	24,000	18,000	24,300
Bolts, area at root of thread......................	10,000	15,000	11,250	15,200
Axial compression:				
Axial compression, gross section..................	$\frac{16,000}{1+\frac{1}{13,500}(L/r)^2}$	$\frac{24,000}{1+\frac{1}{13,500}(L/r)^2}$	$\frac{18,000}{1+\frac{1}{13,500}(L/r)^2}$	$\frac{24,300}{1+\frac{1}{13,500}(L/r)^2}$
but not to exceed the value for L/r = 40				
L= length of the member, in inches				
r= least radius of gyration of the member in inches				
Compression splice material, gross section.........	16,000	24,000	18,000	24,300
Bending on extreme fiber:				
Compression in flanges of beams and plate girders..	$\frac{16,000}{1+\frac{1}{2,000}(L/b)^2}$	$\frac{24,000}{1+\frac{1}{2,000}(L/b)^2}$	$\frac{18,000}{1+\frac{1}{2,000}(L/b)^2}$	$\frac{24,300}{1+\frac{1}{2,000}(L/b)^2}$
L= length in inches of the unsupported flange between lateral connections or knee braces				
b= flange width, in inches				
Tension in rolled shapes, built sections and girders, net sections...........................	16,000	24,000	18,000	24,300
Pins..	24,000	36,000	27,000	36,500
Diagonal tension:				
In webs of girders and rolled beams, at sections where maximum shear and bending occur simultaneously	16,000	24,000	18,000	24,300
Shear:				
Girder webs, gross section.........................	10,000	15,000	11,250	15,200
Pins and shop driven rivets........................	12,000	18,000	13,500	18,250
Power driven field rivets and turned bolts.........	10,000	15,000	11,250	15,200
Hand driven rivets and unfinished bolts............	8,000	12,000	9,000	12,150
Bearing:				
Pins, steel parts in contact and shop driven rivets.	24,000	36,000	27,000	36,500
Power driven field rivets and turned bolts.........	20,000	30,000	22,500	30,400
Hand driven rivets and unfinished bolts............	16,000	24,000	18,000	24,300
Expansion rollers, pounds per linear inch..........	600d	900d	675d	910d
d = diameter of roller in inches				

2. Dead and Live Load Stresses

The change in dead load which has occurred since the bridge was constructed, in addition to the specification changes in live and wind loading stated previously, required that the loads and stresses in various members be computed and compared with the specified allowables. See Table II, following, for the summary of the results of the stress comparison.

As shown in Table II, the stresses in the floor stringers are still below allowable levels, in spite of the increase in dead load and presently specified live loads they carry.

The floorbeams in Span 5 show a three per cent overstress. This overstress is insignificant and no strengthening of the floorbeams is required. Floorbeams in the remaining spans are stressed to approximately 95 per cent of the allowable stress. The stress in floorbeams in Spans 1, 2, and 3 does not exceed the allowable stress as reported in the 1969 Condition Report of the Sagamore Bridge. This is due to the use of light weight concrete in the 1963 repairs of the deck of the Bourne Bridge, while in the repairs to the deck of the Sagamore Bridge normal weight concrete was used.

TABLE II
STRESS REVIEW

STRINGER NO. 8		DEAD LOAD	LIVE LOAD	IMPACT	D+L+I	ALLOWABLE	D+2(LL+I)	ALLOWABLE	YIELD STRENGTH
SPANS 1, 2, & 3									
	(1931	6,000	9,800	2,850	18,650	24,300	31,300	32,400	45,000
	(1969	6,780	10,900	3,200	20,880	24,300			
SPANS 4 & 7	(1931	3,280	7,500	2,200	12,980	18,000	22,680	24,000	33,000
	(1969	3,710	7,800	2,300	13,810	18,000			
SPAN 5	(1931	3,580	7,200	2,100	12,880	18,000	22,180	24,000	33,000
	(1969	4,040	7,500	2,200	13,740	18,000			
SPAN 6	(1931	2,840	7,250	2,200	12,290	18,000	21,740	24,000	33,000
	(1969	3,210	7,600	2,300	13,110	18,000			

TABLE II (CONT'D)
STRESS REVIEW

TYPICAL FLOORBEAM		DEAD LOAD	LIVE LOAD	IMPACT	D+L+I	ALLOWABLE	D+2(LL+I)	ALLOWABLE	YIELD STRENGTH
SPANS 1, 2, & 3									
Web Shear	(1931	5,450	2,190	510	8,150	11,250	10,850	15,000	33,000
	(1969	5,950	3,300	790	10,040	11,000			
Flange Bending	(1931	11,600	7,500	1,770	20,870	24,300	30,140	32,400	45,000
	(1969	11,600	10,000	2,400	24,000	24,300			
SPANS 4 & 7									
Web Shear	(1931	3,400	2,550	690	6,640	11,250	9,880	15,000	33,000
	(1969	3,560	3,460	960	7.980	11,000			
Flange Bending	(1931	10,100	7,500	2,000	19,600	24,300	29,100	32,400	45,000
	(1969	10,100	10,500	2,940	23,540	24,300			
SPAN 5									
Web Shear	(1931	3,850	2,670	690	7,210	11,250	10,570	15,000	33,000
	(1969	4,020	3,510	980	8,510	11,000			
Flange Bending	(1931	11,150	7,950	2,050	21,150	24,300	31,150	32,400	45,000
	(1969	11,150	10,800	3,050	25,000	24,300			
SPAN 6									
Web Shear	(1931	3,570	2,890	820	7,280	11,250	10,990	15,000	33,000
	(1969	3,740	3,690	1,050	8,480	11,000			
Flange Bending	(1931	10,150	8,160	2,290	20,600	24,300	31,050	32,400	45,000
	(1969	10,150	10,500	2,980	23,630	24,300			

The increase in load on the hanger cables as a result of the current dead, live, and impact loads, has decreased the original factor of safety, based on the yield strength of the material, from 2.45 to 2.15. This reduction of approximately 12 per cent still leaves an adequate factor of safety and should not cause concern.

Truss members are subject to an increase in dead load of approximately 3 per cent as a result of the 1963 deck repairs. Current governing live load on the truss is the HS20 lane loading which is approximately 10 per cent less than the lane load used in the original design. The net effect of an increase in dead load and a specification decrease in live load is a combined load slightly less (3 per cent) or equal to that used in the original design. The difference being in the ratio of dead to live load for any particular member. The results of the traffic studies indicate that the current computed lane loading is considerably less than the design lane loadings compared above.

3. Wind Stresses

Members carrying wind loads were investigated due to the difference between the 1931 and 1969 design wind loading specified.

The current combined loading of wind on live load and wind on structure for the design of the suspended floor bracing in Span 1 is less than that specified in the 1931 specifications. The suspended floor bracing is therefore adequate for the currently specified loads.

The 1969 AASHO specified loading for the design of the truss bracing and portals is greater than that specified in 1931. The increase in specified wind load on structure is 66 per cent for Span 1 and approximately 35 per cent for combined wind on structure and wind on live load in the remaining spans. The difference in increase being that in Span 1, the wind on live load is carried by the suspended floor bracing. Since the wind on live load specified in 1969 is only 50 per cent of that specified in 1931, the combination of these two forces in Spans 2 through 7 produces less of an increase in wind stress than in Span 1 in which the truss bracing members carry wind on structure only. The allowable stresses specified for lateral loads provides a factor of safety of 1.83 based on the yield strength of the material. For bracing members that may be stressed right up to the original allowable limit, an increase in load of 66 per cent would cause a decrease in factor of safety to 1.1 and an increase in load of 35 per cent would cause a decrease in factor of safety to 1.35.

Considering that the structure has withstood several hurricanes without distress, and that the current maximum design wind loading is conservative, the strengthening of the wind bracing would be a conservative, but not a necessary procedure. As pointed out previously, the need for maintaining the wind bracing in "like new" condition is imperative. During future inspections, particular attention should be paid to the condition of these members.

The increase wind loading does not cause stresses above allowables in the truss members, as the wind loading is not a large portion of the total stress in these members and because, currently, a 25 per cent increase in stress is permissible when wind load is combined with dead load.

VIBRATION MEASUREMENTS

Significant vibrations of the structure, at different locations along the roadway deck, were measured instrumentally under loading of heavy truck traffic. Seismographic measurements were made by Weston Geophysical Research, Inc., of Weston, Massachusetts, under the direction of Dr. F. Thomas Turcotte. A report of the data obtained from the vibration study is included as Appendix V of this report.

The largest amplitude of traffic induced vibration measured was 0.49 inches at a frequency of 0.11 cycles per second. This low frequency was observed only on Span 5. The transmission of vibration from one span to the next indicates a more rigid contact between spans on the north end of Span 1 than at the south end. There was no evidence of any sympathetic periodic vibrations being built up at any point in the structure as a result of live loads. The live load deflections measured by survey instruments are included in the Profile Survey section of this report.

FAY, SPOFFORD & THORNDIKE, INC.
ENGINEERS
BOSTON, MASS.

APPENDIX I

1971 CONDITION REPORT
BOURNE HIGHWAY BRIDGE

PHOTOGRAPHS

(Two Books Under Separate Cover)

APPENDIX II

SUMMARY OF SUPERSTRUCTURE INSPECTION NOTES

1971 CONDITION REPORT
BOURNE HIGHWAY BRIDGE

TRUSS MEMBERS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1				
L10'E-L11'E	Lacing	Surface Rust	IB-23R	
L14'E-U15'E	Outside Face	Surface Rust	IB-25R	
L16'W-U15'W	Lacing	Mod. Rust	IB-26L	
U15E-L16E	Lacing	Heavy Rust to 100%	IB-26R	B3
L10'E-L11'E	Inside L10'E Conn.	100 Sq. In. 1/4" Deep Rust	IB-41L	
SPAN 2				
U8'W-U9'W	Lacing Near U9'W	Rusting to 50%	IC-14L	
L8'E-U7'E	Lacing	Surface Rust	IC-14R	B7
L8'E-L7'E	Lacing	Surface Rust	IC-14R	
L6'E-U7'E	40% of Lacing	Mod. Rust	IC-15L	
L5'E-U5'E	Top Gusset	Mod. Rust	IC-15R	
U5'E-L4'E	Lacing	Rust to 25%	IC-16L	
L3'E-L2'E	Lacing	Mod. Rust	IC-17R	
L2'E-U1'E	Lacing Rivets	12-25%	IC-18L	
L0'W-U0'W	Full Length	Mod. Rust	IC-18R	B11
L0'E-U0'E	At Jacking Beam	4 Rivets 25%	IC-20R	
Jacking Beam@0'	Rivets E. End	6 Rivets 25%	IC-20R	
L0'E-U0'E	Inside of Member	Moderate Rusting	IC-27R	B8
SPAN 3				
L7W-U7W	Top East Face	Surface Rust	IB-10R	
L4W-U4W	Improper Cleaning of Member		IA-52L	B13

TRUSS MEMBERS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 4				
L8E-U8E	Full Length	Surface Rust	IB-30R	
L1W-L0W	Full Length	Surface Rust	IB-32R	
U1W-L0W	Full Length	Surface Rust	IB-32R	
U5W-U6W	Full Length	Paint Poor	IB-35L	
U6E-U7E	Full Length	Paint Poor	IB-35L	
SPAN 5				
L0W-U0W	Paint	Poor Condition	IIA-11L	
SPAN 6				
U7E-L6E	Paint Peeling	Surface Rust	IB-18R	
U7E-U6E	Inside Plates	Surface Rust	IB-18R	
L7E-L6E	Bot. Lacing	Surface Rust	IB-18R	
L1W-L0W	S. End, Bot. of Top Pl.	Rust--100 sq. in. x 1/16"-1/8" Deep	IB-21L	
SPAN 7				
U3W-L4W	Paint	Flaking Off	IC-4R	
U5W-L6W	Inside W. Face	Rust to 3"x3"x 1/4"	IC-5R	
U7W-L8W	Near Top Conn.	Surface Rust	IB-14R	
L8W-U8W	Paint	Poor Condition	IIA-11L	B12

TRUSS BRACING CONNECTIONS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1				
U11'E	W. Gusset, Pl.	Mod. Rust 10 sq.in.	IB-23R	
U16'W	Bot. Gusset, Pl.	Mod. Rust	IB-26L	
U15'E	Bot. Gusset, Pl.	Mod. Rust	IB-25L	
U14W	E. Face	6"x 6" Surface Rust	IB-26R	
L11W	Gusset Rivet	Mod. Rust	IB-28L	
SPAN 2				
L8'E	Gusset Pl.	Surface Rust	IC-14L	
L8'E	Gusset Pl. Rivets	6 of 22 25%	IC-14L	
L8'W	Top Gusset Pl. Rivets	1-75%, 2-25% of 22	IC-14L	
L8'W	Top Gusset Pl. Rivets	1 of 5 75%	IC-14L	
L7'E	Gusset Pl. Rivets	8 Riv. 25% Max 3 of 7 25%	IC-14R	
L7'W	Top Gusset Pl. Rivets	1 of 7, 50% 1 of 7, 75%	IC-15L	
L7'W	Top Gusset Pl.	Mod. Rust 1'x2' area	IC-15L	B19
L6'W	N. Top Gusset Pl.	Mod. Rust	IC-15L	
L6'E	Inside Joint Pl. Rivets	1, 50% 1, 25%	IC-15L	
L6'E	Inside Joint Pl.	Large Areas Surface Rust	IC-15L	
L6'E	Gussets for L6'E-L6'W	Mod. Rust	IC-15L	
L5'E	T and B Gusset	Mod. Surface Rust	IC-15R	
L4'E	Top Gusset Pl.	Heavy Rust, Hole 6"x 6"	IC-16L	B18

TRUSS BRACING CONNECTIONS

	Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 2 (Cont.)					
	L4'E	Top Gusset Pl. Rivets	5 of 50 25%	IC-16L	B18
	L4'E	Bot. Gusset at Edge	Heavy Rust	IC-16L	
	U5'E	Gusset Under FB5'	Mod. Rust	IC-16L	B22
	L3'E	Top Gusset Pl. Rivets	3 of 13, 75%, 5-25%	IC-16R	
	U4'E	Bot. of Gusset to FB4'	Heavy Rust	IC-16R	
	U3'E-S	Top Gusset Rivets	4-25%, 4-50%, of 14	IC-17L	
	U3'E-S	Top Gusset Rivets	Of 9, 1-25%, 2-50%, 1-75%	IC-17L	
	U3'E-S	Top Gusset	Mod. Rust	IC-17L	
	U4'E-N	Top Gusset	Heavy Rust, Corner Bent Up	IC-17L	
	U4'E-N	Top Gusset Rivets	Of 14, 3-75%, 4-25%, 1-50%	IC-17L	
	L2'E	Top Gusset	Mod. Rust	IC-17R	
	L2'E	Inside Joint Lower Chord	4 Rivets 25%	IC-17R	
	U2'E-S	Top Gusset Rivets	Of 14; 1-75%, 2-50%, 3-25%	IC-17R	B23
	U2'E-S	Top Gusset Rivets	Of 9; 3-25 %; of 7; 1-25%	IC-17R	B23
	U2'E-S	Top Gusset	Mod. Surface Rust	IC-17R	B23
	U2'E-S	Bot. Gusset	Edge Rusted, Holes	IC-17R	
	U2'E-S	Bot. Gusset Rivets	Of 11; 5-75%, 1-25%	IC-18L	
	U2'W-S	Bot. Gusset	Mod. Rust	IC-18L	
	L0'W	Angle to Wind Brace	Of 24; 3-75%, 8-25%	IC-18R	
	U4'E-S	Top Gusset	Heavy Rust	IC-19L	

TRUSS BRACING CONNECTIONS

	Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 2 (Cont.)					
	U4'E-S	Top Gusset Rivets	1-O.K.; Rest 50-100%, of 14	IC-19L	B24
	U4'E-S	Side Gusset	Very Heavy Rust	IC-19L	
	U4'E-S	Side Gusset Rivets	5 of 23 50%	IC-19L	
	U4'E-S	Bot. Gusset	Large Holes 75%	IC-19L	B26
	U4'E-S	Bot. Gusset Rivets	Of 14; 2-75%, 2-50%	IC-19L	B26
	U5'E-N	Top Gusset Rivets	Of 14;1-50%, 3-25%	IC-19R	
	U5'E-N	Top Gusset Rivets	Of 9;3-25%	IC-19R	
	U1'E-S	Top Gusset	Mod. Rust	IC-19R	
	U1'E-S	Top Gusset Rivets	Of 30;15-25%, 5-50%	IC-19R	
	U1'E-S	Bot. Gusset	Edges Eaten 1" in	IC-19R	
	U1'E-S	Bot. Gusset Rivets	Of 30;6-75%, 10-25%, 5-50%	IC-19R	
	U2'E-N	Gusset Rivets	Of 14;2-50%, 3-25%	IC-19R	
	U2'E-N	Top&Bot. Gussets	Mod. Rust	IC-19R	
	L0'E	Gusset Plates	Mod. Rust	IC-20L	B20
	U0'E	T and B Gusset Pls.	Mod. Rust	IC-20L	B31
	U0'E	Bot. Gusset Plates Rivets	2-50%	IC-20L	
	L0'E	Gusset to Jacking Beam Rivets	All to 25%	IC-20R	
	L0'E	Inside Member	Water Sitting	IC-20R	
SPAN 3					
	U0E-N	Upper Gusset Pl. Rivets	17 of 35,25-50%	IB-7L	
	U0E-N	Lower Gusset Pl. Rivets	17 of 30, 25-50%	IB-7L	
	U1E-N	Lower Gusset Pl.	2" 100% W. Edge	IB-7R	

TRUSS BRACING CONNECTIONS

SPAN 3 (Cont.)

Member	Affected Part	%Loss of Material	Field Book Reference	Photograph Reference
U1E-N	Lower Gusset Pl.	Surface Rusted to 1/4"	IB-7R	
U1E-N	Lower Gusset Pl. Rivets	1 of 14-100%	IB-7R	
U1E-N	Top Gusset Pl. Rivets	Of 14, 1-75%, 1-50%	IB-7R	
U1E-N	Top Gusset Pl.	2-4" Areas-1/4"Deep	IB-7R	
U3E-S	Top Gusset Pl. Rivets	3 of 9-25%	IB-8L	
U3E-S	Bot. Gusset Pl. Rivets	3-50%, 1-25%, of 9	IB-8L	
U2E-N	Bot. Gusset Pl. Rivets	5 of 7, 50%	IB-8L	
U2E-N	Top Gusset Pl. Rivets	4 of 14 50%	IB-8L	
U2E-N	Top Gusset Pl. Rivets	4 of 7 50%	IB-8L	
U4E-N	Top Gusset Pl. Rivets	15 of 33 50-75%	IB-8R	B28
U4E-N	Bot. Gusset Pl.	20-30 sq. in. lost at edge	IB-8R	B27
U4E-N	Bot. Gusset Pl. Rivets	5 of 14 50-75%	IB-8R	
L0E	Paint	Poor Condition	IB-8R	
L2E	Top Gusset Pl. Rivets	2 of 14 50%	IB-9L	
L2E	Bot. Gusset Pl. Rivets	2 of 7 50%	IB-9L	
L4E	Top Gusset Pl.	6"x1/2" on Edge 100%	IB-9L	
L5E	Top Gusset Pl.	4"x2" Hole	IB-9L	
L5E	Bot. Gusset Pl.	1/4" Rust	IB-9R	
L6E	Top Gusset Pl.	Hole 15 sq. in.	IB-9R	

TRUSS BRACING CONNECTIONS

	Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 3 (Cont.)					
	L6E	Bot. Gusset Pl.	Hole 4 sq. in.	IB-9R	
	U6E-N	Gusset Pl. Rivets	9 of 12 - 50%	IB-9R	
	L6W	Top Gusset Pl. Rivets	16 of 63 - 25%	IB-10L	
	L6W	Top Gusset Pl.	Edge Loss 3 sq. in.	IB-10L	
	L8E	Bot. Gusset Pl.	Edge Loss 2"x 6"	IB-10L	
	L9W	Inside Bearing Gussets	Surface Rust	IB-11R	A112
	U3E-N	Bot. Gusset Pl.	Edge Loss 3"-4", Only 1/8" to 1/4" of Pl. Left	IA-51R	B25
	U3E-N	Top Gusset Pl. Rivets	10 of 14 50%	IA-51R	B25
SPAN 4					
	U8E-S	Gusset Pl.	Mod. Rust	IB-22L	
	U6E-S	Gusset Pl.	Mod. Rust	IB-31L	
	U6E-S	Gusset Pl. Rivets	5 of 30 25%	IB-31L	
	L3W	Gusset Pl.	Edge Rust 1 sq. in.	IB-32L	
SPAN 5					
	U0E	Gusset Plate	Moderate Rust	IB-5, 15L	
	U8E	Gusset Plate	Heavy Rust to 1/8"	IB-16L	
	U8W	Gusset Rivets	4 of 8 - 25%	IB-16R	B30
SPAN 6					
	U6E-N	Gusset Pl. Rivets	7 of 20 25%	IB-19L	
	U2E-N	Gusset Pl.	Bent Down	IB-20R	
	U2W-N	Gusset Pl.	Bent Down	IB-20R	
	U2E-S	Gusset Pl.	Mod. Rust	IB-20R	

TRUSS BRACING CONNECTIONS

	Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 3 (Cont.)					
	L6E	Bot. Gusset Pl.	Hole 4 sq. in.	IB-9R	
	U6E-N	Gusset Pl. Rivets	9 of 12 - 50%	IB-9R	
	L6W	Top Gusset Pl. Rivets	16 of 63 - 25%	IB-10L	
	L6W	Top Gusset Pl.	Edge Loss 3 sq. in.	IB-10L	
	L8E	Bot. Gusset Pl.	Edge Loss 2"x 6"	IB-10L	
	L9W	Inside Bearing Gussets	Surface Rust	IB-11R	A112
	U3E-N	Bot. Gusset Pl.	Edge Loss 3"-4", Only 1/8" to 1/4" of Pl. Left	IA-51R	B25
	U3E-N	Top Gusset Pl. Rivets	10 of 14 50%	IA-51R	B25
SPAN 4					
	U8E-S	Gusset Pl.	Mod. Rust	IB-22L	
	U6E-S	Gusset Pl.	Mod. Rust	IB-31L	
	U6E-S	Gusset Pl. Rivets	5 of 30 25%	IB-31L	
	L3W	Gusset Pl.	Edge Rust 1 sq. in.	IB-32L	
SPAN 5					
	U0E	Gusset Plate	Moderate Rust	IB-5, 15L	
	U8E	Gusset Plate	Heavy Rust to 1/8"	IB-16L	
	U8W	Gusset Rivets	4 of 8 - 25%	IB-16R	B30
SPAN 6					
	U6E-N	Gusset Pl. Rivets	7 of 20 25%	IB-19L	
	U2E-N	Gusset Pl.	Bent Down	IB-20R	
	U2W-N	Gusset Pl.	Bent Down	IB-20R	
	U2E-S	Gusset Pl.	Mod. Rust	IB-20R	

TRUSS BRACING CONNECTIONS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 7				
L2W	Bot. Gusset Pl.	5"x 1/2"x 1/4" Deep	IC-3R	
U0E	Gusset Pl.	Surface Rust	IC-3R	
L0E-L0W	Gusset at Midspan	Surface Rust	IC-4L	B16
L0E-L0W	Gusset at Midspan Rivets	1 of 20 50% 3 of 20 25%	IC-4L	B16
L3E-L3W	Gusset at Midspan Rivets	9 of 9 25-50%	IC-4R	B17
L4E	Bot. Gusset Pl. Rivets	5 of 6 25-50%	IC-5L	B15A
U4E-S	Gusset Pl. Rivets	2 of 6 25%	IC-5L	
L6W	Gusset Pl.	Surface Rust	IC-5L	
L7E	Inside Vert. Gusset Pl.	Rusting to 1/4"	IC-5R	
L7E-L7W	Gusset at Midspan Rivets	1 of 20 50%	IC-5R	
L0E	Gusset Pl.	Surface Rust	IB-13L	

SUSPENDED FLOOR BRACING CONNECTIONS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1				
FB11W-N	Bot. Gusset Rivets	3 of 12 - 75%	IC-10L	
FB11W-N	Bot. Gusset Rivets	3 of 12 - 50%	IC-10L	
FB11W-N	Top Gusset Rivets	1 of 14 - 75%	IC-10L	
FB11W-N	Top Gusset Rivets	2 of 14 - 25%	IC-10L	
FB11W-N	Top Gusset Rivets	3 of 15 - 75%	IC-10L	
FB11W-N	Top Gusset Rivets	2 of 15 - 50%	IC-10L	
FB-11W-N	Top Gusset Rivets	1 of 15 - 25%	IC-10L	
FB13W-S	Gusset Rivets	1 of 12 - 50%	IC-10R	
FB12E-N	Bot. Gusset Rivets	of 12, 2-75%, 1-25%	IC-10R	
FB13E-N	Top Gusset Rivets	1 of 12 - 25%	IC-11L	
FB15W-S	Gusset Rivets	1 of 12 - 50%	IC-11R	
FB15'W-S	Gusset Rivets	1 of 12 - 25%	IC-12L	
FB16E-N	Bot. Gusset Rivets	of 14, 1-75%, 3-25%	IC-12L	
FB14'W-S	Bot. Gusset Rivets	1-50%, 3-25%	IC-12R	
FB10'W	Vert. Gusset Pl. and Rivets	Heavy Rust	IB-40R	B44
L10'W	End of W.C. Bolt	Snapped off	IB-41R	
L10'E	End of W.C. One Bolt	Snapped Off	IB-41R	
FB13'E-N	Gusset Rivets	1 of 12 - 50%	IB-42R	
FB11E-N	Gusset Rivets	3 of 7 - 75%	IC-10L	

TRUSS BRACING MEMBERS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1				
U9'W-U10'E	Lacing	Heavy Rust to 50%	IB-23L	B47
U9'W-U10'E	Angle	Mod. Rust	IB-23L	B47
U9'W-U10'E	Midspan Rivet	1 of 7--50%	IB-23L	
U11'E-U11'W	Bot. Angles	Heavy Rust	IB-23R	B46
U12'W-U12'E	Full Length	Surface Rust	IB-24L	
U13'W-U14'E	Top	Surface Rust	IB-25L	
U15'W-U15'E	Bot. Busset Pl.	Mod. Rust	IB-25L	B10
L10E-L10W	E. Gusset Pl. Rivets	9 of 40-25%	IB-12L	B50
L10E-L10W	E. End Bot. Side	Heavy Rust	IB-12L	B49
SPAN 2				
L8'E-L8'W	Inside Channel	Surface Rust	IC-14L	
L8'E-L8'W	Cross-Lacing	8 of 60 50-75%	IC-14L	
L8'W-L7'E	Lacing and Plates	Surface Rust	IC-14R	
L7'E-L7'W	Lacing	Heavy Rust Some 75-100%	IC-15L	
L6'E-L6'W	Rivets E. End	6 of 22 25%	IC-15L	
L5'E-L5'W	Lacing	Mod. Rust 2 Members 75%	IC-15R	
L3'E-L3'W	4 Lacing Members	50%	IC-16R	
U4'W-U3'E	Angles at U3'E	Mod. Rust	IC-17L	
L2'E-L2'W	Lacing and Channel	Mod. Rust	IC-17R	
U3'W-U2'E	Angles at U2'E	Bot. Angles 50%	IC-17R	
L1'E-L1'W	Rivets	2 of 40 75%	IC-18L	

TRUSS BRACING MEMBERS

	Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 2 (Cont.)					
	L0'E-U0'W	Lacing	Heavy Rust, Some 100%	IC-18R	B32, B33
	L0'E-L0'W	Lacing	Heavy Rust, Some 50-75%	IC-18R	B40
	U5'W-U4'E	Bot. Angles at U4'E	Very Heavy Rust	IC-19L	
	U1'E-U2'W	Bot. Angles at U1'E	Very Heavy Rust to 3/16"	IC-19R	
	U0'E-U1'W	Bot. Angles at U0'E	Heavy Rust	IC-20L	B42
SPAN 3					
	U0E-L0W	Lacing Bars	1-100% 5-Heavy Rust	IB-7R	
	L9E-L10W	Upper Lacings	Very Rusty	IB-11R	
	L9W-L10E	Upper Lacings	Very Rusty	IB-11R	
	L7E-U7W	Lacing Bars	3-4 50 to 100%	IA-52L	B34
	L7W-U7E	Lacing Bars	3-4 50 to 100%	IA-52L	B34
SPAN 4					
	U8E-U6W	North Corner	Heavy Rust	IB-30R	
	U6E-U8W	North Corner	Heavy Rust	IB-30R	
	L8 W-U8E	Top Face Near Bot.	Mod. Rust	IB-30R	
	L8E-U8W	Top Face Near Bot.	Hole 3 sq. in.	IB-30R	B35
	L8W-L6E	In Bay 8-7	Surface Rust	IB-30R	
	L8E-L6W	In Bay 8-7	Surface Rust	IB-30R	
	U8E-U6W	In Bay 7-6	Mod. Rust Bet. Angles	IB-31L	
	U6E-U8W	In Bay 7-6	Mod. Rust Bet. Angles	IB-31L	
	U6E-U4W	Near F. B. 6	Mod. Rust Bet. Angles	IB-31R	

TRUSS BRACING MEMBERS

	Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 4 (Cont.)					
	U4E-U6W	Near F.B. 6	Mod. Rust Bet. Angles	IB-31R	
	U0W-U2E	Near F.B. 0	Heavy Rust Bet. Angles	IB-32R	
SPAN 5					
	L6E-L8W	West Side	Heavy Rust	IB-4, 6L	
	L2E-L4W	Full Length	Mod. to Heavy	IB-5	
	U0W-U2E	West Side	Heavy Rust	IB-5R	B36
	U6E-U8W	50% of Length	Mod. Rust	IB-16L	
	U6E-U8W	West End	1/4" Deep Rust	IB-16R	B39
SPAN 6					
	L8E-L8W	Top Leg	Mod. Rust	IB-17R	
	L8E-L6W	East Side	Heavy Rust	IB-18L	
	U8E-U6W	Full Length	Bowed	IB-18L	
	U6E-U8W	Full Length	Bowed	IB-19L	
	U6E-U8W	End Gussets	Slight Downward Bend	IB-19L	
	U8E-U6W	End Gussets	Slight Downward Bend	IB-19L	
	L6W-L5W	Lacing Bars	Mod. Rust	IA-33L	
SPAN 7					
	L6E-L8W	75% of Length	Surface Rust	IC-5R	
	L6W-L8E	50% of Length	Surface Rust	IB-12R	
	L0E-L2W	50% of Length	Surface Rust	IB-13L	
	L0E-L2W	East End	Mod. to Heavy Rust	IA-49R	B38
	U2E-U4W	Full Length	Member Bowed	IA-49R	B41
	U6E-U8W	Lower Angle	Resting on Catwalk	IA-49R	

SUSPENDED FLOOR BRACING MEMBERS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1				
BAY 11-12	W. Wind Chord Rivets	1/4th - 0-25%	IC-9R	
BAY 12-13	W. Wind Chord Rivets	4 - 25%	IC-10R	
BAY 12-13	W. Wind Chord	Mod. Rust on Plate	IC-10R	
W14-E13	Bet. Angles	Rusting to 1/4"	IC-11L	
BAY 13-14	W.C. Conn to FB14	Surface rust	IC-11L	B43
BAY 13-14	W.C. West, Rivets	3 - 25%	IC-11L	
BAY 14-15@ FB14	W.C. West, Rivets	of 12, 2 - 50%, 1 - 25%	IC-11R	
BAY 15-16	W.C. West, Rivets	2 - 25%	IC-11R	
BAY 16-15'	W.C. West, Rivets	4 - 25%, 2 - 50%	IC-12L	
W15'-E16	Bot. Fl.	1' to 2' of rust	IC-12L	
BAY 16-15'	W.C. East @ FB15'	2 sq. ft. rust	IC-12L	B128
FB15'E	Pl. under cable ends	Mod. rust to 1/8"	IC-12R	B127
FB14'E	W.C. Rivets	1 - 25%	IC-12R	
BAY 10'-11'	N. end of W.W.C.	10 rivets 50%	IB-49R	B51
BAY 9'-10'	W.C. Rivets at FB10'E	8 - 50%	IB-41R	B52
BAY 9-10	E.W.C. Slots	Heavy Rust	IB-12L	
E15'-W14'	1 of 2 vertical legs	25%	IA-53L	

SIDEWALK AND CURB SUPPORTS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1				
Bay 10-11	Knee brace to curb channel	1 bolt missing	IC-7R	B62
Bay 10-11	Knee brace to curb channel	Top Plate 25%	IC-7R	B62
Bay 10-11	Pl. Supp. curb	Mod. Rust	IC-7R	B62
Bay 10-11	SW Bolts at FB10	Heavy Rust	IC-9R	
FB10	SW Angles	Mod. Rust to 3/16"	IB-48R	B64
Bay 13-14	SW Bolt Heads at FB13	OF 4; 1-75-100%	IC-11L	
AT FB14'	SW Channel Nut	25 to 50%	IC-12R	
FB13'	SW Channel Nuts	2 Heavy Rust	IB-43L	
Bay 10'-11'	SW Channel at M. H.	One Spot Rusted to 1/4"	IB-49R	
Bay 10'-11'	Gusset at M. H.	Mod. Rust	IB-49R	
Bay 10'-11'	Angle at M. H.	50-75% Loss of horiz. leg	IB-49R	
Bay 10'-11'	SW Channel Rivets to FB10'	OF 4; 1-75%, 1-50%	IB-49R	
SPAN 2				
Bay 0'-1'	Angle @ FBO' and S. W. Chan.	Heavy Rust Angle Twisted	IB-50R	B56
Bay 0'-1'	Vert. Pl. E. of S. W. Chan.	Mod. to Heavy Rust	IB-50R	B57
Bay 5'-4'	Pl. Supporting curb.	Heavy Rust	IC-16L	B63
Bay 5'-4'	Pl. Supporting curb.	One Nut Loose, 50% Rusted	IC-16L	B63
Bay 4'-3'	Curb channel @ 3'	Heavy Rust		
Bay 3'-2'	Pl. BETW ST9 & Curb channel	Mod. Rust	IC-18L	
Bay 1'-0'	Gap betw. FBO' and curb plate	Heavy Leakage	IC-20L	B61

SIDEWALK AND CURB SUPPORTS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 3				
Bay 0-1	SW Channel Nuts @ FBO	Heavy Rust	IB-16R	B53
Joint @ Pier 3	SW Channel 1 Nut	90%	IB-16R	B59
Bay 0-1 @ FBO	SW Channel Nuts	4 of 4 - 50 to 75%	IB-6R	
Joint @ Pier 3, FBO	SW Channel Nuts	2 of 8 - 100%	IB-7L	B54
Joint @ Pier 3, FBO	SW Channel Nuts	2 of 8 - 50%	IB-7L	
Joint @ Pier 3, FBO	SW Channel	Heavy Rust	IB-7L	
Bay 0-1	SW Chan. 10' from FBO	Heavy Rust	IB-7L	
Bay 0-1@FBO	SW Chan. @ STI	Heavy Rust	IA-51R	
SPAN 4				
Bay 0-1	Angles at M. H.	Mod. to Heavy Rust	IB-50L	
Bay 0-1	Pl. Bet. STI and SW	Mod. Rust @ FBO	IB-50L	B55
SPAN 6				
Bay 7-8	Curb Channel @ FB8	Heavy Rust	IB-17R	
Bay 0-1	Angle at M. H. Under SW	75% Horiz. Leg.	IB-49L	B60
Bay 0-1	Of 4 Nuts at M. H. SW Channel	1-75%, 1-50%	IB-49L	B58
SPAN 7				
Joint @ S. Abut.	SW Channel	1/4" Loss	IC-4	
Bay 7-8	Top Fl. SW Channel @ FB8	Rusting to 1/8" Deep	IB-47R	
Bay 7-8	Pl next to SW Channel	Heavy Rust to 1/4"	IB-48L	

FLOOR BEAMS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1				
FB10	Top Fl. E. End	Heavy Rust to 25%	IC-7R	
FB10	E. End Gusset	2 of 2 Rivets 50%	IC-7R	
FB10	At St. 1	Top Fl. Heavy Rust	IC-9L	B83
FB12-N	W. End Rivets	1 of 12 - 50%	IC-10R	
FB12-N	Pl. at F.B. Bet.St. 1+2	Heavy Rust	IC-10R	
FB16-N	Bet. St. 8+9	5' Surface Rust	IC-12L	
FB10'-S	Bet. St. 8+9	Top Fl. 4 of 15 Bolts Missing	IB-41L	B84
FB10'-S	Bet. St. 7+8	Top Fl. 4 of 15 Bolts Missing	IB-41L	
FB10'-S	WEB, S. Side	Surface Rust	IB-41L	
FB10'-S	Bet. St. 6+7	Top Fl. 2 Bolts Missing	IB-41L	
FB10'-S	Full Length	Bolts Missing Top Fl.	IB-41L	
FB10'-S	Bot. Fl. Underside	Surface Rust	IB-41R	
FB10'-S	E. End Rivets	8 - 50%	IB-41R	
FB10'-S	St. 1 Bracket Rivets	9 of 16 25%	IA-53R	
FB10'-S	Top Fl. at St. 1	Mod. Rust	IB-49R	
FB11'E-S	Bot. Fl. Rivets	5 of 12 - 25%	IB-42L	
FB13'E-N	Top Fl. Edge	Mod. Rust	IB-42R	B80
FB13'W-S	Top Fl.	1 sq. ft. Surface Rust	IB-43L	
FB13'E-S	Top Fl.	Mod. Rust	IB-43L	B78
FB13'E-S	Bot. Fl.	Mod. Rust	IB-43L	
FB10W	Underside Bot. Fl.	Mod. Rust	IB-12L	

FLOOR BEAMS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 2				
FB5'	E. End Bot. Fl.	Mod. Rust	IC-16L	B22
FB5'	E. End Rivets	Of 13,3-25%, 2-50%, 4-75%	IC-16L	B22
FB3'	Top Fl. E. End	Heavy Rust	IC-17L	
FB3'	Bot. Rivets E. End	Of 3;1-25%, 2-50%	IC-17L	
FB4'	Bot. Fl. Rivets E. End	Of 7;3-75%, 1-100%	IC-19L	
FB4'	Bot. Fl. Rivets E. End	Of 9;8-75-100%	IC-19L	
FB0'	Betw. SW and ST 1	Mod. Rust	IC-20L	
FB0'	Bot. Fl. 1/2 Length	Mod. Rust	IC-20L	
FB0'	@ST5 W.	Surface Rust	IB-34R	
FB0'	@Curb Plate	Mod. Rust and Gap	IC-20L	B61
FB4'	E. End Top Flg.	Scale Painted Over		B81
SPAN 3				
FB0	East End,N. Top Fl.	Rust	IB-7L	
SPAN 4				
FB8	Stiffener @ Str. 1	Mod. Rust	IB-49L	B66
FB8	Stiffener @ Str. 1 Rivets	6 of 9 25-50%	IB-49L	B66, B68
FB8	Stiffeners at E. End	Heavy to Mod. Rust	IB-49L	B65
FB8	Pl. Supporting Conc.	Heavy Rust	IB-49L	B65, B68
FB2	Bot. Fl. East End Rivets	5 of 6 25-50%	IB-32R	B79
FB0	E. End Pl. Supporting Conc.	Heavy Rust	IB-33L	B67
FB0	E. End Pl. Stiffener	Heavy Rust	IB-33L	B67
FB0	W. End Stiffener	Heavy Rust	IB-33L	
FB8	Catwalk Rivets	5 of 20, 50-75%	IA-15L	

FLOOR BEAMS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 5				
FB0	S. E. Corner	7" T.+B. Mod. Rust	IB-15L	B70
FB0	Stiffener @SW	Mod. Rust	IB-48L	
FB8	Flange @ST5	Mod. Rust	IA-50L	
FB8	Flange Rivets @ ST5	10 Rivets 25%	IA-50L	B72
FB8	Stiffener on North Side	Mod. Rust	IA-50R	
FB8	Rivets at Catwalk	2 - 50%	IA-50R	
FB8-N	Stiffener @ ST 1	Heavy Rust	IA-51R	B73
FB0	West End	Leakage 8'-10' From End	IB-45L	B71
SPAN 6				
FB8	Stiffener @ ST1	Heavy Rust	IB-17R	B76, B74
FB8	Stiffener @ ST1 Rivets	Heavy rust	IB-17R	
FB8	Splash Pl.	Surface Rust	IB-17R	
FB8	Top Fl. East Side	Mod. Rust	IB-17R	
FB8	North Face East Side	Heavy Rust	IB-17R	
FB6	Top Fl. East Side	Heavy Rust	IB-18R	
FB4	Bot. Fl. East Side	Paint Flaking	IB-19R	
FB3	Bot. Fl. East Side	Rust	IB-20L	
FB0	Stiffener @ E. End	Mod. Rust	IB-21R	
FB0	Conc. Haunch Support	Full Length Mod. to Heavy Rust	IB-21R	B77
FB8	Stiffener at E. End	Heavy Rust	IA-56R	B75

FLOOR BEAMS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 7				
FB0	Haun. Support Full Length	Rust	IC-4L	
FB0	Haun. Support @ ST1	1/2" Edge Loss	IC-4L	B82
FB8	East South Side	1 of 6 rivets 25%	IC-6L	
FB8	Catwalk Rivets	Of 12; 1-50%, 1-25%	IA-10R	

STRINGERS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1				
Bay 10-11	St. 9 @ FB10	Bot. Fl. Bolt heads rusted, one 25%, one 50%, no movement evident. Top fl. mod. rust	IC-7R	B105
Bay 10-11	St. 8 @ FB10	Sliding bolts heavy rust	IC-8L	
Bay 10-11	St. 8 @ FB10	Rivet next to bolt - 50%	IC-8L	
Bay 10-11	St. 7 @ FB10	Bolt heads - 25%	IC-8L	
Bay 10-11	St. 7 @ FB10	Underside of bolts rusted	IC-8L	
Bay 10-11	St. 6 @ FB10	Bolts rusted solid	IC-8R	
Bay 10-11	St. 3 @ FB10	Rivet Head 75%	IC-9L	
Bay 10-11	St. 2-E @ FB10	Rivet Head 75%	IC-9L	
Bay 10-11	St. 5,4,3, 2,1 @ FB10	Bolts rusted to Knee Brace	IC-9L	
Bay 10-11	St. 1 @ FB10	1 bolt missing	IC-9L	
Bay 10-11	St. 1 @ FB10	1 of 7 rivets 75%	IC-9L	
Bay 10-11	St. 2-W @ FB10	1 rivet 100% @ Knee Brace	IC-9L	
Bay 10-11	St. 1 @ FB10	2 of 2 rivets 75 and 100%	IC-9L	
Bay 12-13	St. 1 @ FB12	Top Fl. Heavy Rust	IC-10R	
Bay 16-15'	St. 9 @ FB16	10' surface rust	IC-12L	
Bay 10'-11'	St. 9 @ FB10'	Bot. Fl., 2 nuts heavy rust	IB-23R	
Bay 10'-11'	St. 9 @ FB10'	Slots for bolts heavy rust	IB-23R	

STRINGERS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1 (Cont'd)				
Bay 10'-11'	St. 9 @ FB10'	Knee brace heavy rust	IB-40R	B87
Bay 10'-11'	St. 9 @ FB10'	Plates and rivets heavy rust	IB-40R	B86
Bay 10'-11'	St. 9 @ FB10'	Heavy rust to 1/8"	IB-41L	B107
Bay 10'-11'	St. 1 @ FB10'	Mod. rust bot. fl. and knee brace	IB-41L	B85
Bay 10'-11'	St. 1 @ FB10'	Heavy rust conn. diaphragm	IA-53R	B108
Bay 10'-11'	St. 1 @ FB10'	Top Fl. heavy rust	IA-53R	
Bay 9-10	St. 3 Full Length	Bot. fl. pitting to 1/4"	IA-52L	
Bay 9-10	St. 1 Full Length	Bot. fl. pitting to 1/4"	IA-52L	
SPAN 2				
Bay 6'-5'	St. 9 @ FB5'	Surface rust	IC-15R	
Bay 5'-4'	St. 9 @ FB5'	Top & Bot. fl. Mod. rust	IC-16L	B63
Bay 4'-3'	St. 9	Bent	IC-17L	
Bay 1'-0'	St. 1 @ FB0'	Conn. angle heavy rust	IC-20L	B91
Bay 1'-0'	St. 9 @ FB0'	Mod. Rust	IC-20L	B31, B89
Bay 1'-0'	St. 8 @ FB0'	Mod. Rust	IC-20R	
Bay 1'-0'	St. 5 @ FB0'	Web Heavy rust	IA-54R	B90
SPAN 3				
Bay 0-1	St. 1 Top Fl.	Mod. Rust @ FBO	IA-51R, 31R	B94, B96
Bay 8-9	St. 1 Bot. Fl.	Mod. Rust full length	IA-52L	

STRINGERS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 4				
Bay 8-7	St. 9 @ FB8	T & B fl. Mod. rust	IB-22L	B98, B100
Bay 5-4	St. 1 @ FB5	T & B fl. light rust	IB-31R	
Bay 0-1	St. 1 @ FB0	Top fl. W. side surface rust	IB-50L	B55
Bay 1-2	St. 1 and St. 9	Bot. Fl. heavy pitting to 50%	IA-55L	
Bay 1-2	St. 1 @ FB2	Bot. 4" of Web-50% small holes	IA-55L	
Bay 2-3	St. 9 @ FB3	Bot. 4" of Web-50%	IA-55R	
Bay 2-3	St. 1 @ FB3	Bot. 4" of Web-50% small holes	IA-55R	B101
Bay 3-4	St. 1 and St. 9 @ FB4	Bot. fl. 50%	IA-55R	
Bay 3-4	St. 1 @ FB4	Web Bot. 4"-50%	IA-55R	
Bay 4-5	St. 1 @ FB5	Web Bot. 4"-50%	IA-55R	
Bay 6-5	St. 9 @ FB6	Web Bot. 4"-50%	IA-55R	
Bay 6-7	St. 9 @ FB7	Bot. Fl. Pitted to 50%	IA-55R	
Bay 7-8	St. 9 @ FB8	Bot. 4" of Web, Bot. fl. 50%, small holes	IA-55R	
SPAN 5				
Bay 7-8	N. end St. 5	T & B Fl. Heavy rust	IA-50L, 31L	
SPAN 6				
Bay 7-8	St. 9 @ FB8	T & B fl. Mod. rust	IB-17R	B99
Bay 6-5	St. 9 @ FB6	Heavy rust	IB-18R	B97
Bay 4-3	St. 1 @ FB4	Bot. fl. 10" mod. rust	IB-19R	

STRINGERS

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 6 (Cont'd)				
Bay 3-2	St. 9 Full length	Surface rust	IB-20L	
Bay 2-3	St. 9 @ FB3	Web-Bot. 4" 50%	IA-56L	
Bay 2-3	St. 9 @ FB3	Bot. fl. 50%	IA-56L	
Bay 3-4	St. 1 @ FB4	Web-Bot. 4" 50%	IA-56L	
Bay 3-4	St. 1 @ FB4	Bot. fl. 50%	IA-56L	
Bay 3-4	St. 9 @ FB4	Web-Bot. 4" 50%, holes	IA-56L	
Bay 3-4	St. 9 @ FB4	Bot. fl. 50%	IA-56L	
Bay 5-6	St. 9 @ FB6	Web-Bot. 4" 50%	IA-56L	
Bay 5-6	St. 9 @ FB6	Bot. fl. 50%	IA-56L	
Bay 6-7	St. 1 @ FB7	Web-Bot. 4" 50%	IA-56R	
Bay 6-7	St. 1 @ FB7	Bot. fl. 50%	IA-56R	
Bay 6-7	St. 1 Center 6'	Pitted to 50%	IA-56R	

UNDERSIDE OF CONCRETE DECK

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 1				
Bay 10-11	At FB. 11 and Str. 7	8"x5"x5" spall	IC-9L	
Bay 11-12	At Str. 1	6"x2" spall w/rebar exposed	IC-10L	
Bay 11-12	At FB. 12 and Str. 5, 6, 7, 8	Typ. haunch spall	IC-10L	
Bay 12-13	At Str. 5	Spall full length, rebar exposed	IC-10R	
Bay 12-13	At FB. 12 betw. str. 7 & 8	Haunch spall	IC-10R	
Bay 13-14	At FB. 13 betw. str. 7 & 8	Haunch spall 2' long	IC-11L	
Bay 14-15	At FB. 14 betwn str. 2 & 3	Small spalls, rebars exposed	IC-11R	
Bay 14-15	At str. 9 near FB. 15	Spall and rebar exposed	IC-11R	
Bay 15-16	At FB. 15 betw. str. 6 & 7	Haunch spall	IC-11R	
Bay 15-16	At str. 1 west side 10' from FB. 15	Two large spalls	IC-11R	
Bay 10-11	At M. H.	Spalling w/rebars exposed	IB-48R	B64
Bay 10-11	At M. H.	Honeycomb at conduit (exposed)	IB-48R	
Bay 16-15'	Betw. str. 2 & 3	Small spall (3") w/ rebars exposed	IC-12L	
Bay 15'-14'	Betw. str. 4 & 5 at 20' from FB. 15'	2 ft. of Cracking & spalling	IC-12R	B119
Bay 12'-11'	At str. 1 at 20' from FB. 11'	Small Spall, rebar exposed	IC-13L	
Bay 11'-10'	Along SW Channel	Spalls almost full length	IC-13R	B122

UNDERSIDE OF CONCRETE DECK

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 2				
Joint Pier 4	At exp. joint	5"x12"x12" spall	IA-24L	
Bay 1'-2'	At FB. 1' betw. str. 7 & 8	24"x6" spall	IC-19R	
Bay 2'-3'	Betw. str. 5 & 6	Typ. hairline cracks	IC-18L	B116
Bay 3'-4'	At west side str. 3	Small holes w/rebars exposed	IC-17L	
Bay 4'-5'	Betw. str. 5 & 6 at 16' from FB. 4'	8" hole x 1/2" deep	IC-19R	
SPAN 3				
Bay 0-1	At. FB. 1 and str. 7	Spall 15"x6"x4"	IA-22L	
Bay 0-1	At. FB. 0 and str. 1	Leakage thru deck	IA-51R	
Bay 1-2	At str. 6, 5' from FB. 2	Spall, rebars exposed	IA-22L	
Bay 4-5	On str. 3, 10' from FB. 4	Patched area shows further leakage	IA-25R	
SPAN 4				
Bay 6-5	Betw. str. 4 & 5	Long. crack in patch	IA-55R	B118
Pier 4	At M. H.	Rebar exposed 5' length	IB-50L	B121
Pier 4	At M. H. at north end of grating	Bar rusted (50%) for 6"	IB-50L	
Joint Pier 4	At exp. joint	Spalls, concrete poor condition	IB-33L	B115
SPAN 5				
Bay 0-1	9' from FB. 0	Transv. Crack Full Deck Width	IA-20R	
Bay 0-1	At FB. 0	12"x7"x5" haunch spalls	IA-20R	
Bay 0-1	At FB. 0	Spall	IA-50L	
Bay 3-4	Betw. str. 4 & 5	Spall and cracks on patch	IA-11L	B117
Bay 4-5	At FB. 4 (on catwalk)	Haunch spall	IA-11L	
Bay 6-7	Over catwalk	Spall rebars exposed	IA-11L	B113

UNDERSIDE OF CONCRETE DECK

Member	Affected Part	% Loss of Material	Field Book Reference	Photograph Reference
SPAN 5 (Cont'd)				
Bay 7-8	At M. H. in S. W.	3'x1' spall	IB-17L	B120
Bay 7-8	S. W. 8' from FB. 8	36"x4" spall	IB-17L	
Bay 7-8	S. W. concrete	Very poor condition	IB-17L	
Bay 7-8	At north end str. 5	Leakage thru deck	IA-50L	
Bay 7-8	For 6' on str. 3E from FB. 8	6' spall w/leakage	IA-50R	
Joint Pier 3	North of FB. 8	Spall at haunch	IA-50R	B114
Joint Pier 3	At haunch near catwalk	Concrete poor condition rebars exposed	IA-11R	
Bay 6-7	At str. 2	Two 6"x6" spalls, one has rebar exposed	IA-21R	
SPAN 6				
Bay 8-7	At FB. 8 All stringers	Typical haunch spall	IA-22R	
SPAN 7				
Bay 3-4	Top flg. of str. 5	6"x6" spall rebars exposed	IA-19R	
Bay 3-4	Betw. str. 7 & 8	Stains indic. leakage	IA-19R	
Bay 6-7	At FB. 6 betw. str. 8 & 9	Spall w/3' of rebar exposed	IA-20L	
Bay 7-8	At FB. 7 (on catwalk)	Spall w/rebars exposed	IA-10L	
Bay 7-8	At FB. 8 under S. W. Channel	2'-0"x8"-5" spall rebar exposed	IB-47R	

FAY, SPOFFORD & THORNDIKE, INC.
ENGINEERS
BOSTON, MASS.

APPENDIX III

1971 CONDITION REPORT
BOURNE HIGHWAY BRIDGE

CABLE X-RAYS AND REPORT

X-ray film furnished separately.

ARNOLD GREENE TESTING LABORATORIES, INC.
EAST NATICK INDUSTRIAL PARK, 6 HURON DRIVE • NATICK, MASS. 01760
AREA CODE 617 • PHONE: 235-7330 • 653-5950

BRANCH LABORATORIES:
SANTURCE, PUERTO RICO TEL. (809) 722-1922
267 PAGE BOULEVARD SPRINGFIELD, MASS. 01104 TEL. (413) 734-6546
98 PARIS STREET EVERETT, MASS. 02149 TEL. (617) 387-3770
2 MILLBURY STREET AUBURN, MASS. 01501 TEL. (617) 832-5500
100 PIONEER AVE. WARWICK, RHODE ISLAND 02888 TEL. (401) 467-2666

Ref. No. 10929-96202

June 11, 1971

Fay, Spofford & Thorndike
11 Beacon Street
Boston, Massachusetts 02108

Attention: Mr. Albright

Gentlemen:

Reference is made to the inspection of Suspender Cables on the Bourne Highway Bridge.

On June 2, 3 and 4, 1971, we radiographed selected areas of cable as specified by your representative, Mr. Harrington. These areas were assigned Serial Numbers 12 East-South, 14 East-North, 16 East-North, 14 East-South, 12 East-North, 12 West-North, 14 West-North, 14 West-South, 12 West-South and 16 North-West. These descriptions were supplied by your representatives at the Bridge. All cables were radiographed with two exposures labled 0°and 90° to indicate source and film location in relation to the cable. In addition, the third exposure was made and designated "bottom" to indicate this area as being immediately above the heavy socket fitting at the bottom of the cable. This exposure was only made on areas 16 East-North, 14 East-South and 12 West-North.

A radioactive isotope of Cobalt 60 of 67 Curies was used with a focal spot size .16" diameter by 40" long. Two speeds of film were exposed simultaneously in the same film holder in order to compensate for the curvature of the cable and resultant film density change. The two film used were Kodak types AA and M or AA and T sandwiched between .010" lead filter screens. A sensitivity gauge was recorded on each film. A source to film distance of 25" was employed. All film were developed at the job site.

The double film of all exposures were examined, and the make up of the various strands composing the cable were clearly depicted. No evidence of corrosion was evident on the recorded image of the cables.

We trust this is the information desired.

Very truly yours,

ARNOLD GREENE TESTING LABORATORIES, INC.

Robert J. Halliday

Robert J. Halliday
Vice-President

RJH:sgs

APPENDIX IV

1971 CONDITION REPORT
BOURNE HIGHWAY BRIDGE

PREVIOUS REPORTS

1958 AMERICAN BRIDGE CO. INSPECTION AND CONDITION REPORT

1963 DESIGN MEMORANDUM-BOURNE HIGHWAY BRIDGE-MAJOR REHABILITATION

1969 SUBSTRUCTURE SURVEY NOTES BY CORPS OF ENGINEERS

(THESE REPORTS ARE AVAILABLE AT OFFICE OF CORPS OF ENGINEERS)

FAY, SPOFFORD & THORNDIKE, INC.
ENGINEERS
BOSTON, MASS.

APPENDIX V

1971 CONDITION REPORT
BOURNE HIGHWAY BRIDGE

VIBRATION MEASUREMENTS

Report of Vibration Measurements

WESTON GEOPHYSICAL RESEARCH, INC.
POST OFFICE BOX 364
WESTON, MASSACHUSETTS 02193
AREA CODE 617 894-1020

September 24, 1971

Fay, Spofford & Thorndike, Inc.
11 Beacon Street
Boston, Massachusetts 02108

Gentlemen:

Vibration measurements on the Bourne Bridge, Bourne, Massachusetts were conducted on July 14, 1971.

This is a formal presentation of our findings.

Sincerely yours,

WESTON GEOPHYSICAL RESEARCH, INC.

Thomas Turcotte

Dr. F. Thomas Turcotte

FTT:jh

VIBRATION MEASUREMENTS

BOURNE BRIDGE

for

FAY, SPOFFORD & THORNDIKE, INC.

by

WESTON GEOPHYSICAL RESEARCH, INC.

WESTON, MASSACHUSETTS

VIBRATION MEASUREMENTS

BOURNE BRIDGE

TEST PROCEDURES

Vibration measurements were made on the Bourne Bridge on July 14, 1971. The majority of the measurements were recorded on the pedestrain walkway on the west side of the bridge. Measurements were also made on the curbing on the eastern side of the roadway at the center of each span, excluding Spans 2 and 3.

The vibrations measured were induced by heavy trucks or buses moving on the spans containing the seismometers during the recording period. Wind induced vibration was not felt to be a factor, since only a light breeze was blowing during most of the recording period.

The recording location in the tabulated data is identified by span number followed by an E or W to indicate the east or west side of the roadway, then by the panel number. Two recording points, joined by brackets, indicate that recordings taken at these points were made simultaneously with event marks placed on each record to synchronize the time scales. An effort was made to obtain longer period vibration data by filtering the higher frequencies before amplification and then recording on a slower speed recorder. Periods as long as 8.8 seconds were obtained in this manner. The absolute calibration of the seismograph system is lost in the process however, and the amplitude of motion is reported at the end of the tabulated data under the heading of "extended range vibration results", since the numerical displacement values were normalized to the calibrated data.

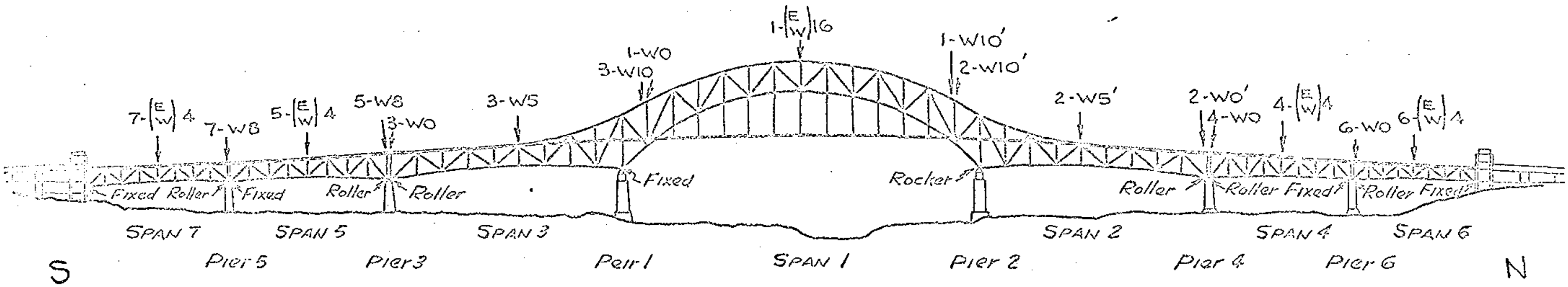
Recordings were made using Sprengnether, three-component DVA-1100 and VS-1100 seismographs. The horizontal component of motion, identified as Longitudinal in the tabulated data, refers to the direction in the length of the bridge (north-south); while the transverse-horizontal component of motion is at right angles to that (east-west). The vibration displacement amplitudes (zero to peak values) are given directly in inches.

COMMENT

The magnitude of the vibration displacements are similar to those found on the Sagamore Bridge in 1969. A comparison of the vertical motion at the center of the three spans of the Sagamore Bridge to the three center spans of the Bourne Bridge shows essentially the same frequency-amplitude pattern on both bridges. Span 2, Panel 5 data are similar on both bridges. Span 1, Panel 16 exhibited larger amplitude motion on the Bourne Bridge in the 1.1 - 2.2 cps range than on the Sagamore Bridge, but is still of the same order of magnitude. Span 3, Panel 5 showed distinctly smaller amplitude at 2.4 cps on the Bourne Bridge than at similar frequencies on the Sagamore Bridge, while in contrast, the 1.7 cps motion was larger on the Bourne Bridge than on the Sagamore Bridge.

The simultaneous recordings on opposite sides of expansion joints permit transmissibility data to be obtained from he records. The longitudinal component of motion was examined across both ends of Span 1. The displacements on Span 1 were larger in every case than the corresponding displacements on Spans 2 or 3. Comparing the end of Span 2, adjacent to the north end of Span 1, we see that the increase is by a factor of 1.1 at 4 cps and by a factor of 1.3 at 12-14 cps. Comparing the end of Span 3 to the south end of Span 1 shows the increase to be by a factor of 2.5 at 1.5 cps and by a factor of 10 at 12 cps. This suggests that the expansion joint is tighter (or possibly frozen) at the north end of Span 1 in contrast to the south end.

The lower frequency motion reported in the "extended range vibration results" table has larger displacements in the vertical direction in all cases. Spans 5 and 7 showed significantly larger vertical motion at low frequencies than elsewhere on the bridge.



(NOTE: Recording Stations are on Roadway Deck)

RECORDING POINTS FOR VIBRATION MEASUREMENTS
BOURNE BRIDGE
CAPE COD CANAL, MASSACHUSETTS
for
FAY, SPOFFORD & THORNDIKE, INC.
by
WESTON GEOPHYSICAL RESEARCH, INC.

BOURNE BRIDGE VIBRATION DISPLACEMENTS

Recording Point	Comment	Vertical Freq. (cps)	Vertical Disp. (in)	Longitudinal Freq. (cps)	Longitudinal Disp. (in)	Transverse Freq. (cps)	Transverse Disp. (in)
6-W4		2.2	.0005	2.2	.0004	2.2	.0011
		4.	.0025	4.	.0006	---	-----
	recorded simultaneously	13.	.0006	13.	.0001	13.	.0002
6-E4		2.5	.0018	2.4	.0006	2.5	.0006
		5.	.0016	4.	.0007	4.	.0002
		----	-----	----	-----	9.	.0004
6-W0		2.4	.0002	2.4	.0004	2.4	.0006
		----	-----	5.	.0020	5.	.0010
		8-11	.0038	----	-----	----	-----
4-W4		1.2	.0042	1.4	.0003	----	-----
		2.0	.0018	----	-----	2.0	.0017
		----	-----	2.2	.00005	2.2	.00005
	recorded simultaneously	5.	.0040	5.	.0007	5.	.0004
		12.	.0014	12.	.0002	12.	.0005
4-E4		----	-----	1.3	.0020	----	-----
		2.1	.0007	2.1	.0007	2.1	.0008
		4.	.0067	6.	.0005	----	-----
		12.	.0011	12.	.0004	10-12	.0006

VIBRATION DISPLACEMENTS (Con't.)

Recording Point	Comment	Vertical Freq. (cps)	Vertical Disp. (in)	Longitudinal Freq. (cps)	Longitudinal Disp. (in)	Transverse Freq. (cps)	Transverse Disp. (in)
4-W0	recorded simultaneously	----	-----	2.0	.0013	----	-----
		----	-----	2.5	.0010	----	-----
		10.	.0043	10.	.0005	10.	.0014
2-W0'		2.2	.0002	2.1	.0013	1.8	.0015
		----	-----	2.5	.0010	----	-----
		10.	.0030	12.	.0004	8-12	.0009
2-W5'		1.4	.0021	1.3	.0063	1.3	.0017
		----	-----	2.3	.0009	2.7	.0008
		4.	.0015	4.	.0007	4.	.0016
2-W10'	recorded simultaneously	1.3	.0014	----	-----	1.9	.0006
		5.	.0011	----	-----	3-4	.0007
		10.	.0007	10.	.0001	----	-----
		13.	.0001	14.	.0003	17.	.0001
1-W10'		1.4	.0027	1.6	.0028	2.2	.0006
		4.	.0016	4.	.0004	3.	.0006
		13.	.0004	13.	.0001	14.	.0002

VIBRATION DISPLACEMENTS (Con't.)

Recording Point	Comment	Vertical Freq. (cps)	Vertical Disp. (in)	Longitudinal Freq. (cps)	Longitudinal Disp. (in)	Transverse Freq. (cps)	Transverse Disp. (in)
1-W16	recorded simultaneously (1-W16 and 1-E16)	----	-----	----	-----	1.1	.0035
		2.2	.0093	----	-----	----	-----
		3.	.0037	----	-----	3.	.0009
		12.	.0002	----	-----	----	-----
1-E16		1.1	.0081	----	-----	1.1	.0024
		----	-----	1.9	.0007	----	-----
		3.	.0025	2.5	.0007	3.	.0010
1-W10	recorded simultaneously (1-W10 and 3-W10)	1.5	.0014	1.5	.0017	1.2	.0016
		2.0	.0007	1.8	.0009	2.3	.0004
		----	-----	----	-----	3.8	.0007
3-W10		----	-----	1.6	.0013	1.6	.0009
		2.2	.0005	----	-----	2.6	.0003
		3.	.0003	----	-----	3.	.0002
		4.	.0003	----	-----	4.	.0004
3-W5		1.7	.0070	1.6	.0014	1.6	.0040
		2.4	.0005	----	-----	----	-----
		10.	.0004	13.	.0001	13.	.0001

VIBRATION DISPLACEMENTS (Con't.)

Recording Point	Comment	Vertical Freq. (cps)	Vertical Disp. (in)	Longitudinal Freq. (cps)	Longitudinal Disp. (in)	Transverse Freq. (cps)	Transverse Disp. (in)
3-W0	recorded simultaneously	----	-----	1.6	.0012	1.6	.0021
		2.4	.0014	----	-----	----	-----
		3.	.0006	----	-----	----	-----
		10.	.0007	10.	.0002	6.	.0012
5-W8		----	-----	1.6	.0013	1.6	.0015
		4.	.0003	3.	.0002	----	-----
		7.	.0013	----	-----	6.	.0002
		10.	.0017	10.	.0002	9.	.0004
5-W4	recorded simultaneously	----	-----	1.7	.0017	1.6	.0019
		2.0	.0021	----	-----	2.2	.0009
		10.	.0010	11.	.0020	10.	.0024
5-E4		----	-----	1.9	.0008	1.7	.0011
		2.3	.0012	2.7	.0004	2.1	.0006
		14.	.0006	----	-----	11.	.0002
7-W8		----	-----	3.	.0003	2.0	.0011
		----	-----	5.	.0004	----	-----
		8-9	.0017	----	-----	8-9	.0006
		11.	.0004	----	-----	10.	.0007

VIBRATION DISPLACEMENTS (Con't.)

Recording Point	Comment	Vertical Freq. (cps)	Vertical Disp. (in)	Longitudinal Freq. (cps)	Longitudinal Disp. (in)	Transverse Freq. (cps)	Transverse Disp. (in)
7-W4	recorded simultaneously	1.8	.0023	2.9	.0045	2.2	.0012
		4.	.0005	4.	.0010	4.	.0006
		8.	.0021	----	-----	10.	.0002
7-E4		2.2	.0012	2.0	.0010	2.2	.0013
		4.	.0055	4.	.0015	4.	.0008
		11.	.0010	----	-----	----	-----

<u>EXTENDED RANGE VIBRATION RESULTS</u>

(Amplitudes Normalized to Data in Previous Table)

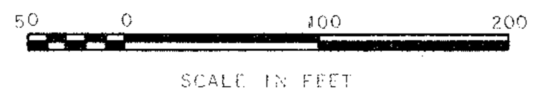
Recording Point	Vertical		Longitudinal		Transverse	
	Freq.	Disp.	Freq.	Disp.	Freq.	Disp.
6-W4	0.25	.016	----	-----	----	-----
	0.50	.032	----	-----	----	-----
	0.63	.014	0.91	.004	0.63	.019
	2.2	.0004	2.2	.0043	2.2	.0011
4-W4	0.30	.0007	0.56	.002	0.6-0.8	.002
	1.0	.0005	----	-----	----	-----
	1.4	.0001	1.2	.0002	1.4	.0004
	2.2	-----	2.2	-----	2.2	.0096
1-W16	0.56	.044	----	-----	----	-----
	----	-----	1.2	.009	1.1	.028
	1.4	.0056	1.4	.0023	1.4	.0016
5-W4	0.11	.490	----	-----	----	-----
	----	-----	1.2	.010	----	-----
	1.4	.006	1.4	.005	1.4	.004
	2.2	.0017	----	-----	2.2	.0009
7-W4	0.26	.105	----	-----	----	-----
	2.2	.0023	1.9		2.2	.0012
	----	-----	2.8	.0045	----	-----

PLATE I

LEGEND

N = NORTH
S = SOUTH
E = EAST
W = WEST
F.B. = FLOOR BEAM
L = LOWER CHORD PANEL POINT
U = UPPER CHORD PANEL POINT
MLW = MEAN LOW WATER BASED ON MSL = 100
ST = STRINGER
SW = SIDEWALK
W.C. = WIND CHORD
M.H. = MAN HOLE

ELEVATION



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

CAPE COD CANAL
BOURNE, MASS.

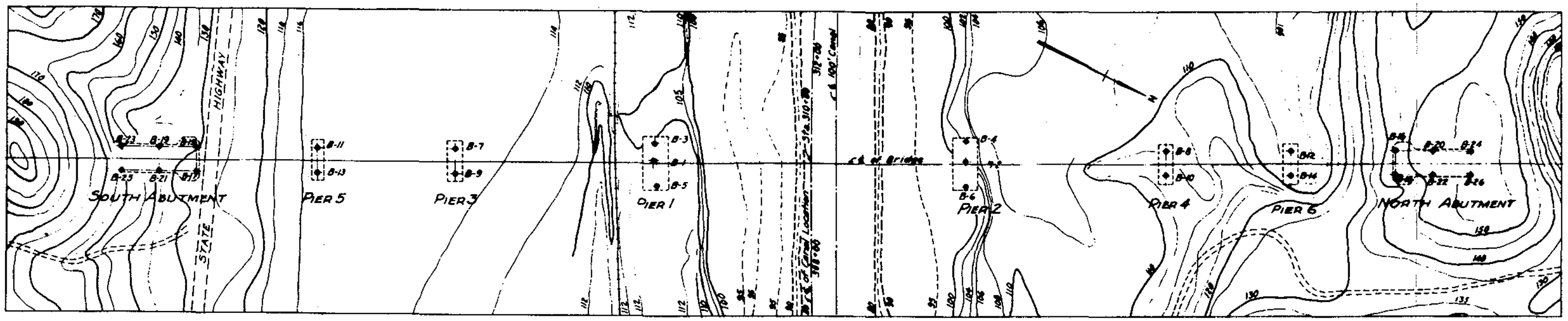
BOURNE HIGHWAY BRIDGE
1971 CONDITION REPORT

KEY PLAN

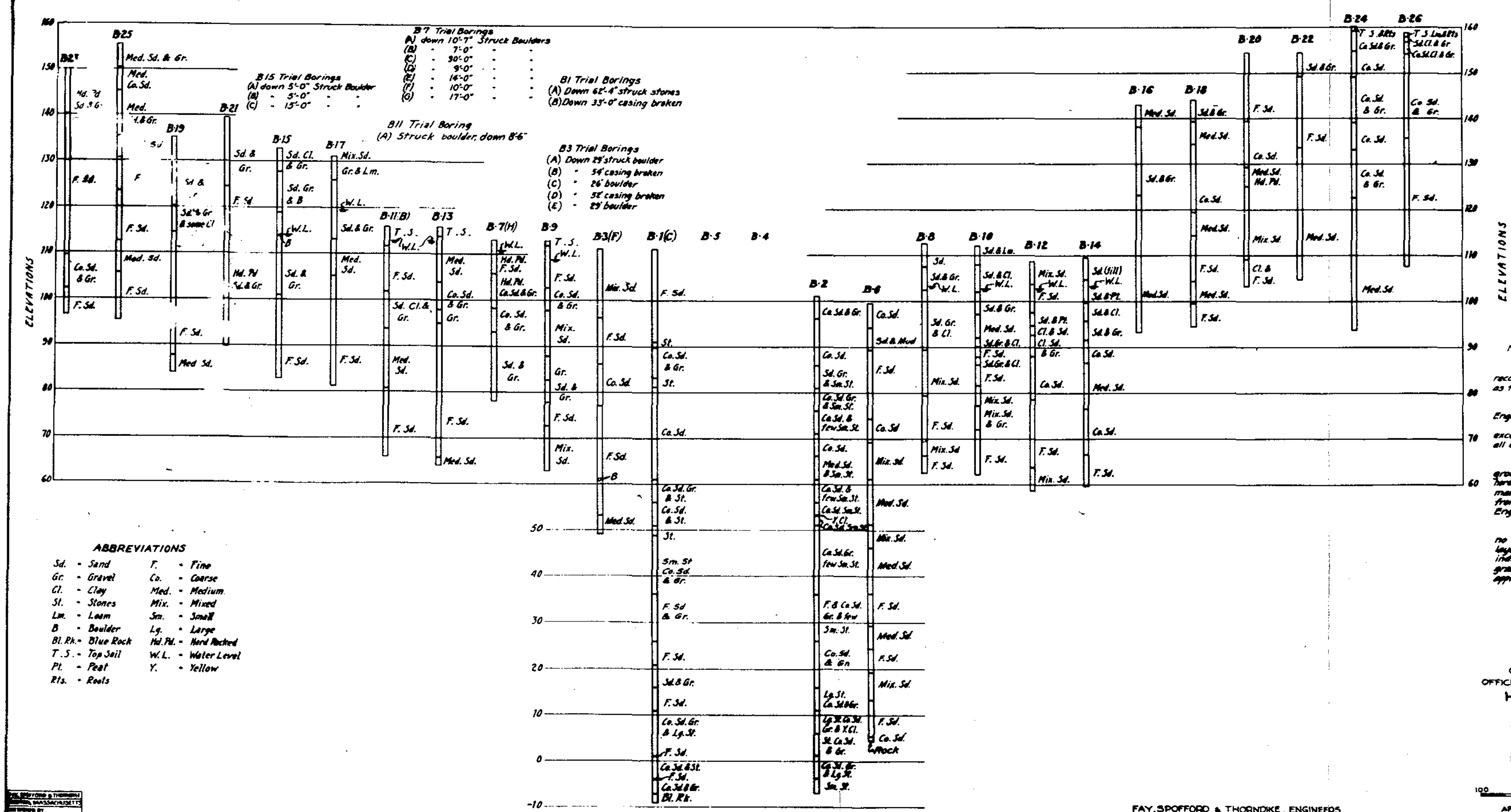
FAY, SPOFFORD & THORNDIKE, INC., BOSTON, MASS.

VL-56
C-3

PLATE II



BORING PLAN



RECORD OF BORINGS

ABBREVIATIONS

Sd.	Sand	F.	Fine
Gr.	Gravel	Co.	Coarse
Cl.	Clay	Med.	Medium
St.	Stones	Mix.	Mixed
Lm.	Loam	Sm.	Small
B	Boulder	Lg.	Large
Bl. Rk.	Blue Rock	Hd. Pd.	Hard Packed
T. S.	Top Soil	W. L.	Water Level
Pt.	Peat	Y.	Yellow
Rts.	Roots		

Notes.

Borings were made by the wash method.

Additional borings are now in progress and the record of these borings may be obtained from the Engineer as the work proceeds.

Samples of Borings may be seen at the U. S. Engineer Office near the Buzzards Bay end of the Canal.

Borings were taken at the locations indicated except for a boring where several trials were made, all of which are within 5 feet of locations shown.

Such records as are available of levels at which ground water was encountered in making borings are here shown. Levels at which wash water was lost while making borings are not here shown but may be obtained from the boring records on file in the office of the Engineer.

Due to apparent lack of stratification, in general no sharp lines of demarcation were found between layers of different materials, and such lines here indicated are to be taken as approximate only. Similarly gradation as to sizes or fineness of materials is approximate only.

FAY, SPOFFORD & THORNDIKE, ENGINEERS

WAR DEPARTMENT
CORPS OF ENGINEERS U. S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.

HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS

BOURNE BRIDGE
RECORD OF BORINGS

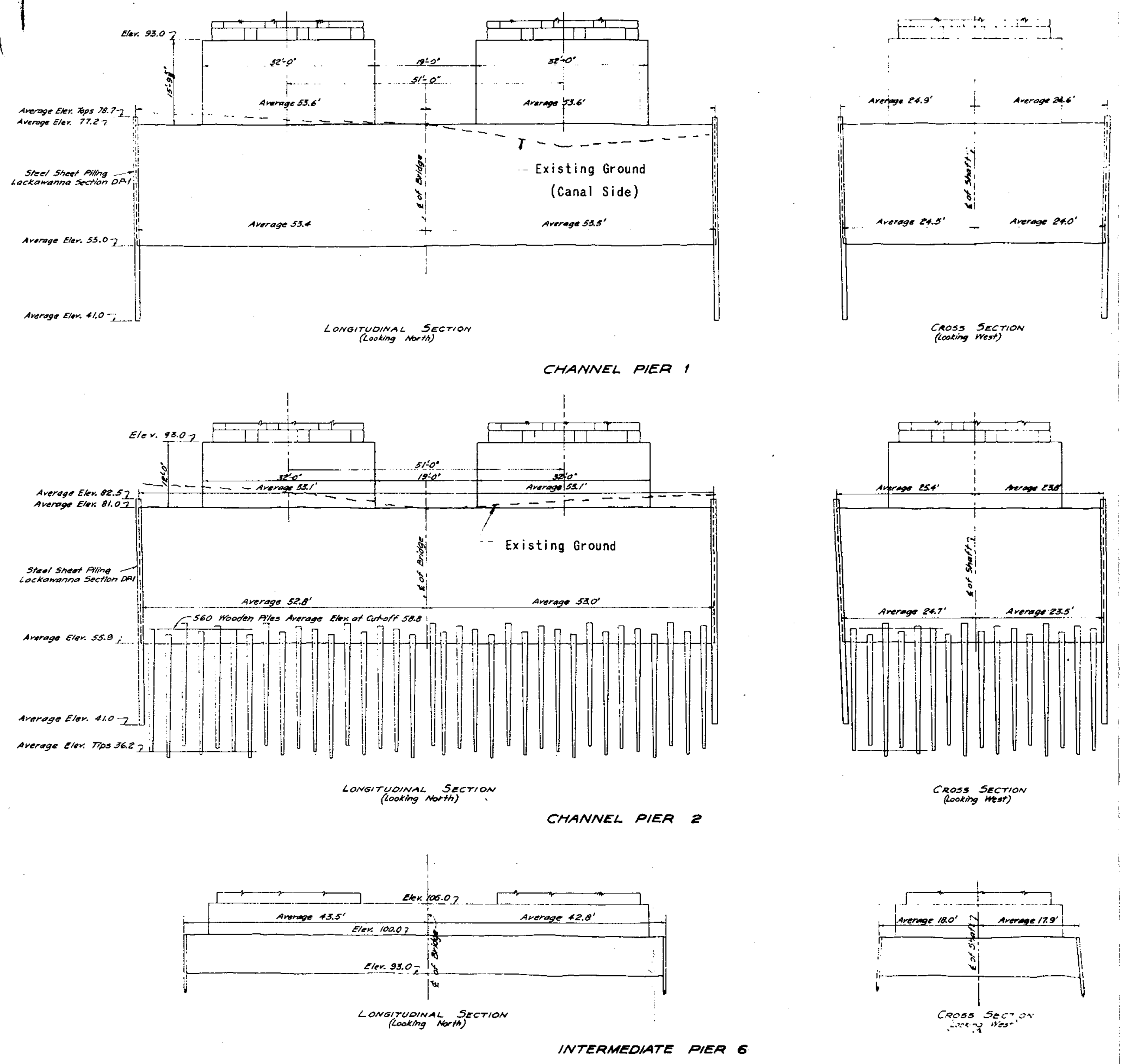
SCALE EXCEPT AS SHOWN
100 0 100 200 300 400 FT.

APPROVED
LT. COL. CORPS OF ENGINEERS
OCTOBER 1933

SUBSTRUCTURE CONTRACT PLANS - SHEET No. 5 OF 19

VL-56

PLATE III



Note - The depths of foundations as built at the other piers and at abutments are as shown on the Contract Drawings. All elevations are referred to a datum 100 feet below Mean Sea Level.

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OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.

HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
BOURNE BRIDGE
FOUNDATIONS AS BUILT
PIERS 1, 2 & 6

SCALE

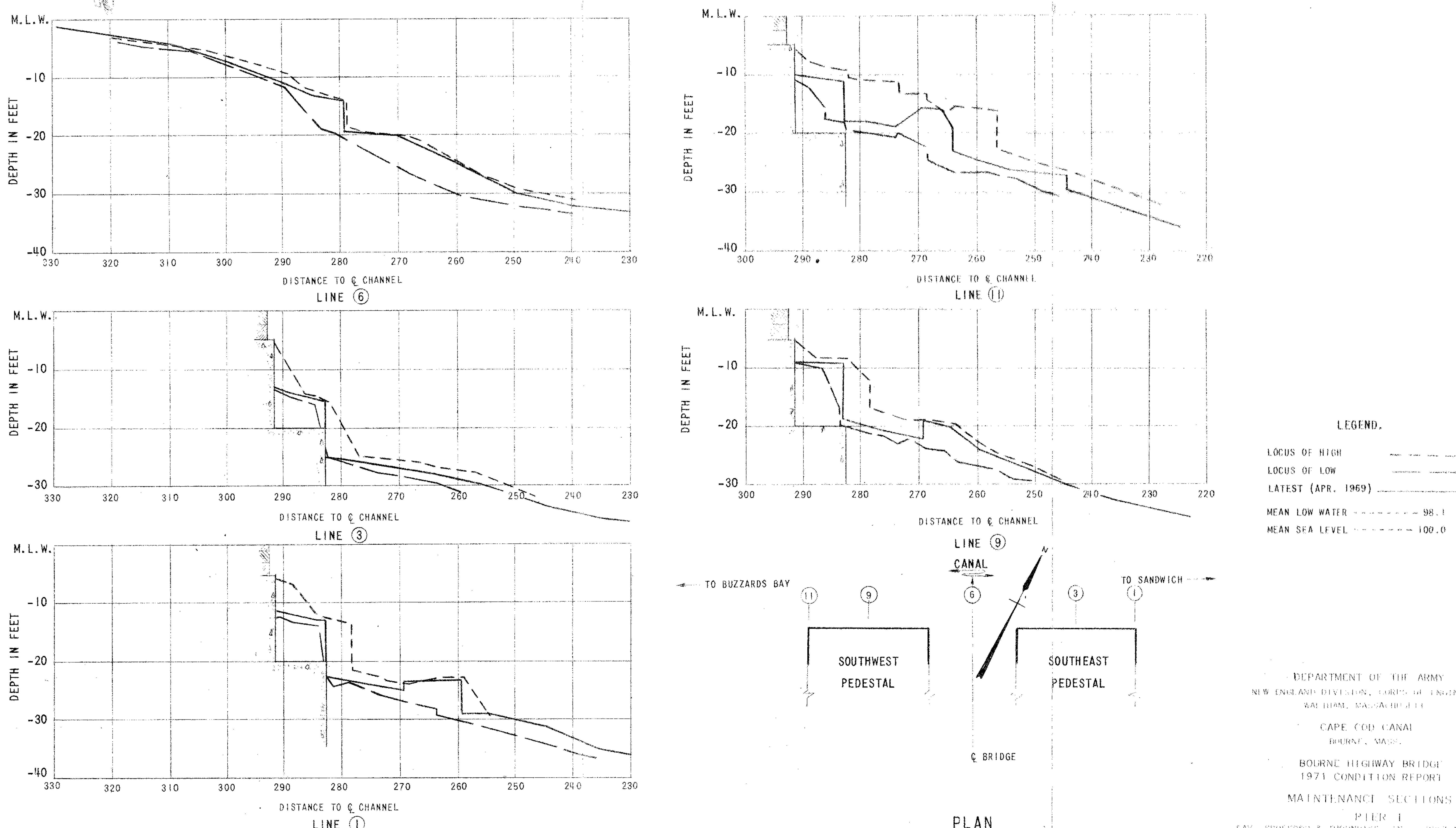
JULY, 1935

FAY, SPOFFORD & THORNDIKE, ENGINEERS
BY ____

FILE 472-1 E-12-1 RECORD PLAN

VL-56

PLATE IV



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

CAPE COD CANAL
BOURNE, MASS.

BOURNE HIGHWAY BRIDGE
1971 CONDITION REPORT

MAINTENANCE SECTIONS
PIER 1

FAY, SPOFFORD & THORNDIKE, INC., BOSTON, MASS.

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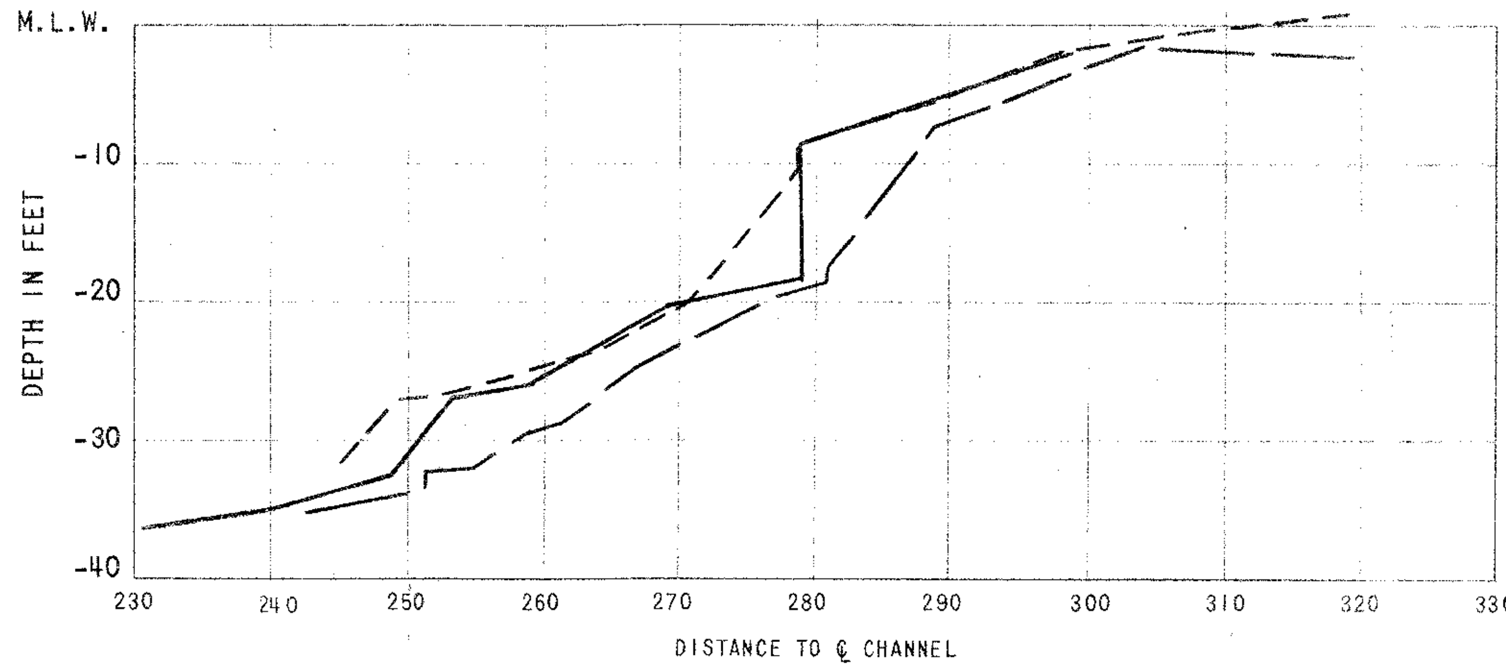
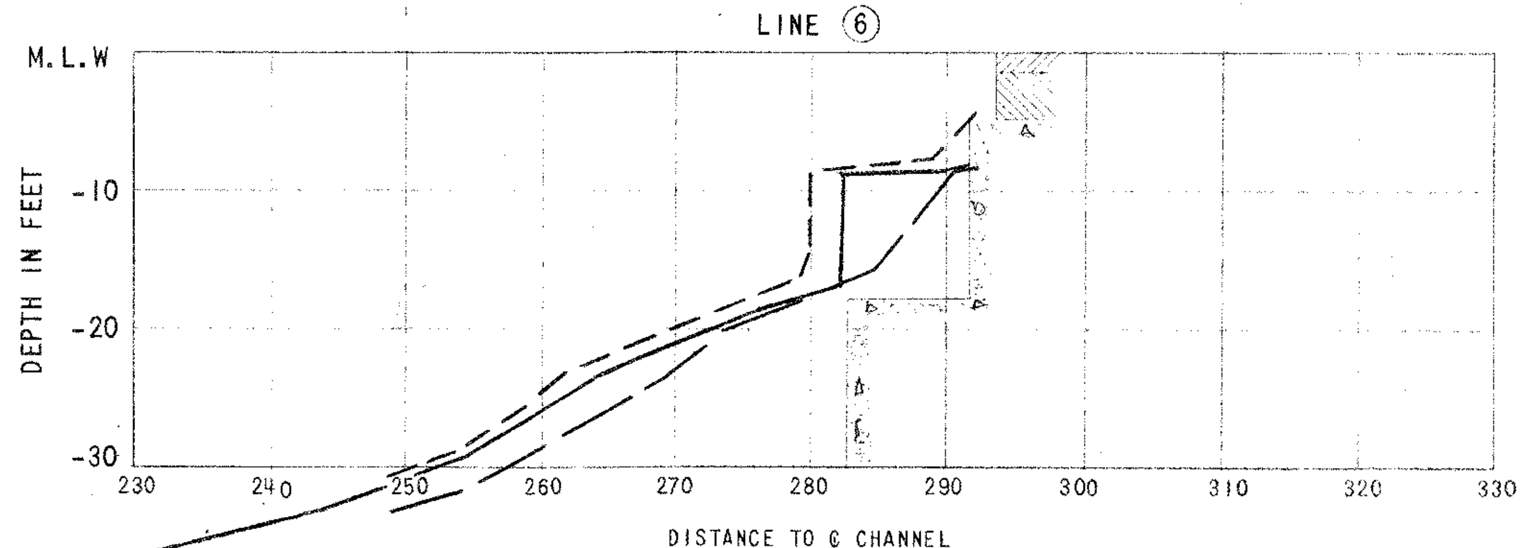
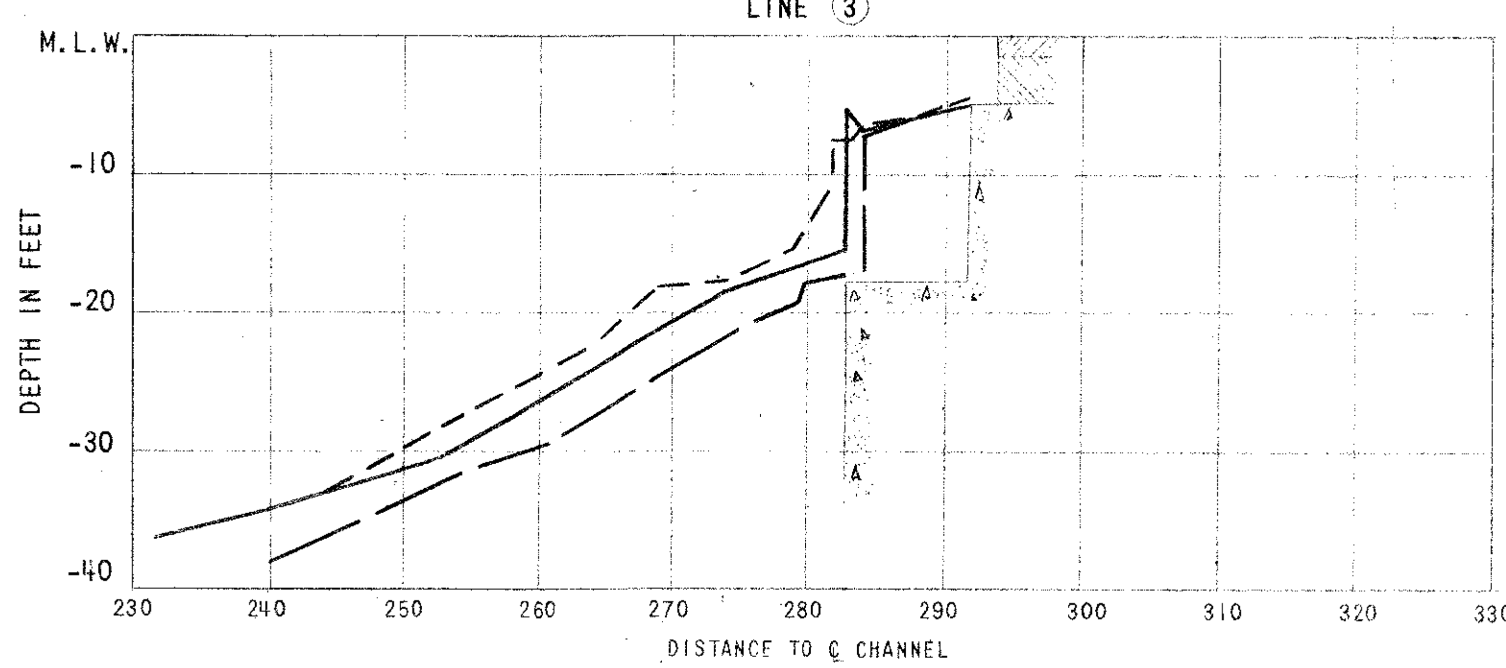
C-6



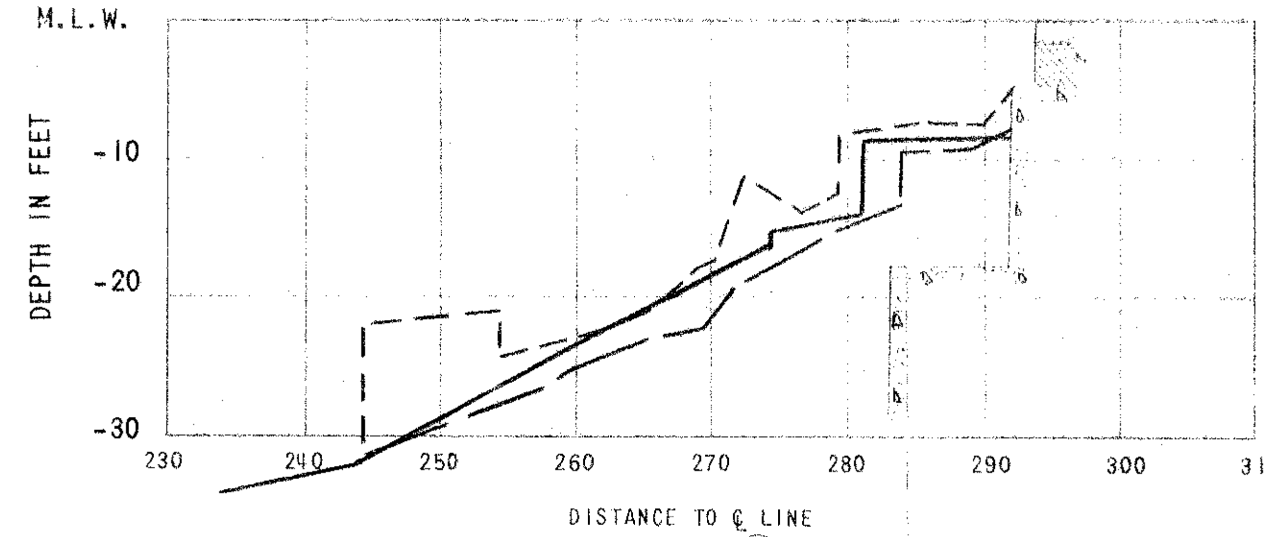
LINE ⑥



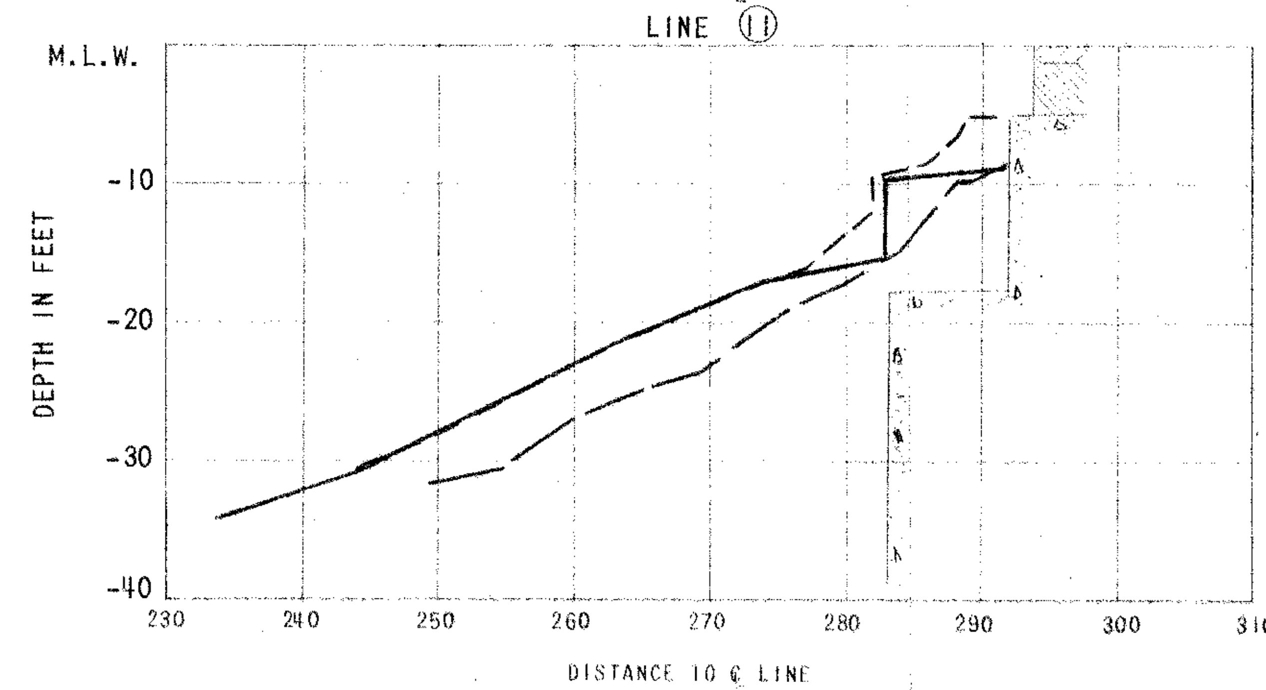
LINE ③



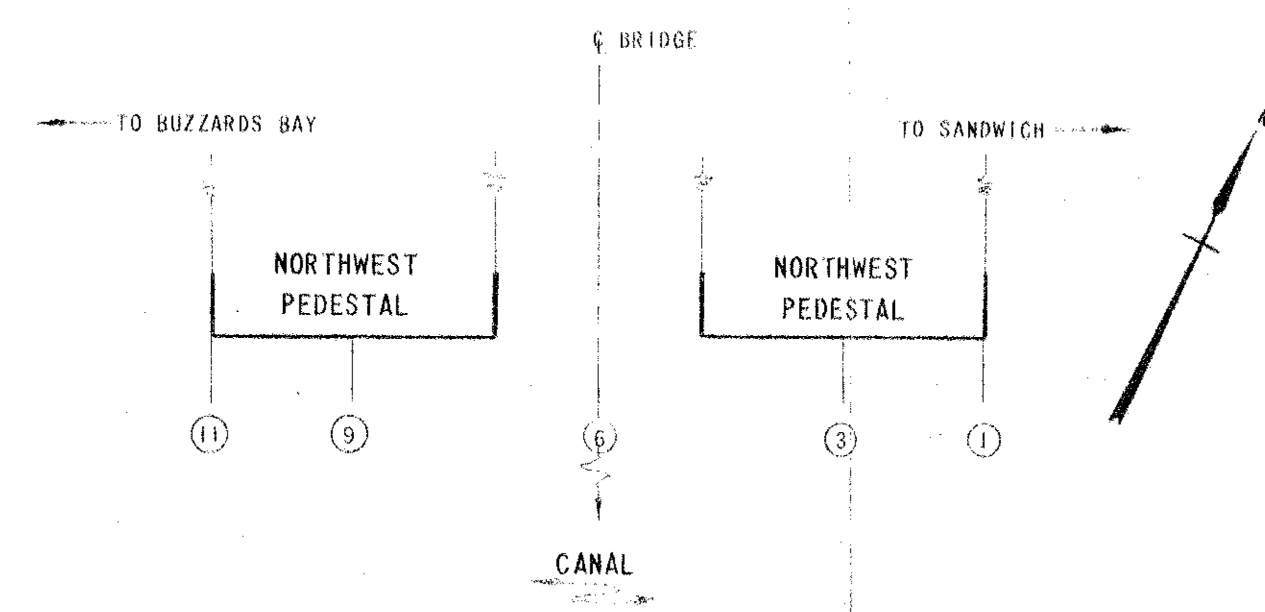
LINE ①



LINE ⑪



LINE ⑨



PLAN

LEGEND

LOCUS OF HIGH
LOCUS OF LOW
LATEST (APR. 1969)
MEAN LOW WATER — 98.1
MEAN SEA LEVEL — 100.0

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

CAPE COD CANAL
BOURNE, MASS.

BOURNE HIGHWAY BRIDGE
1971 CONDITION REPORT

MAINTENANCE SECTIONS
PIER 2

FAY, SPOFFORD & THORNDIKE INC., BOSTON, MASS.

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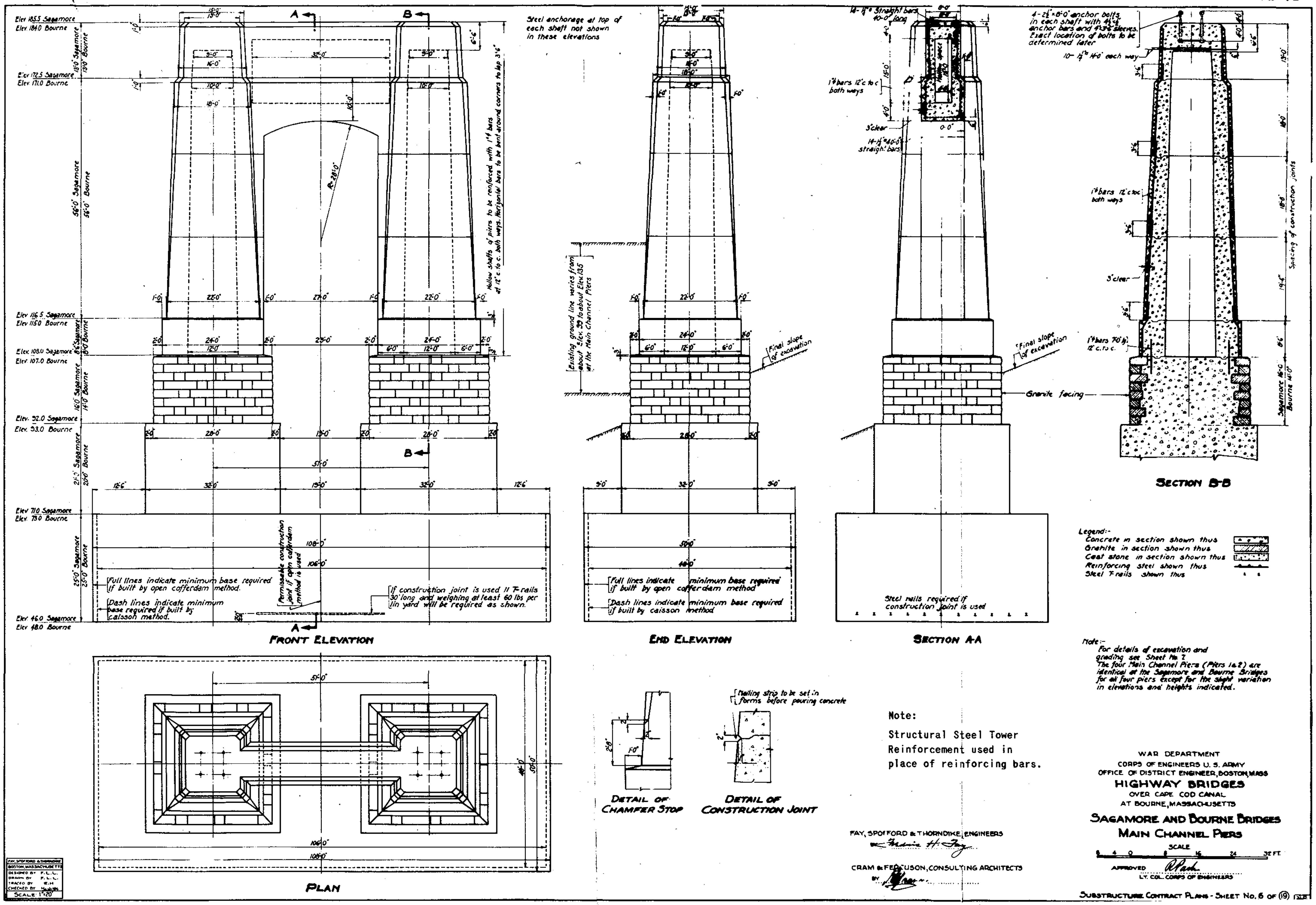
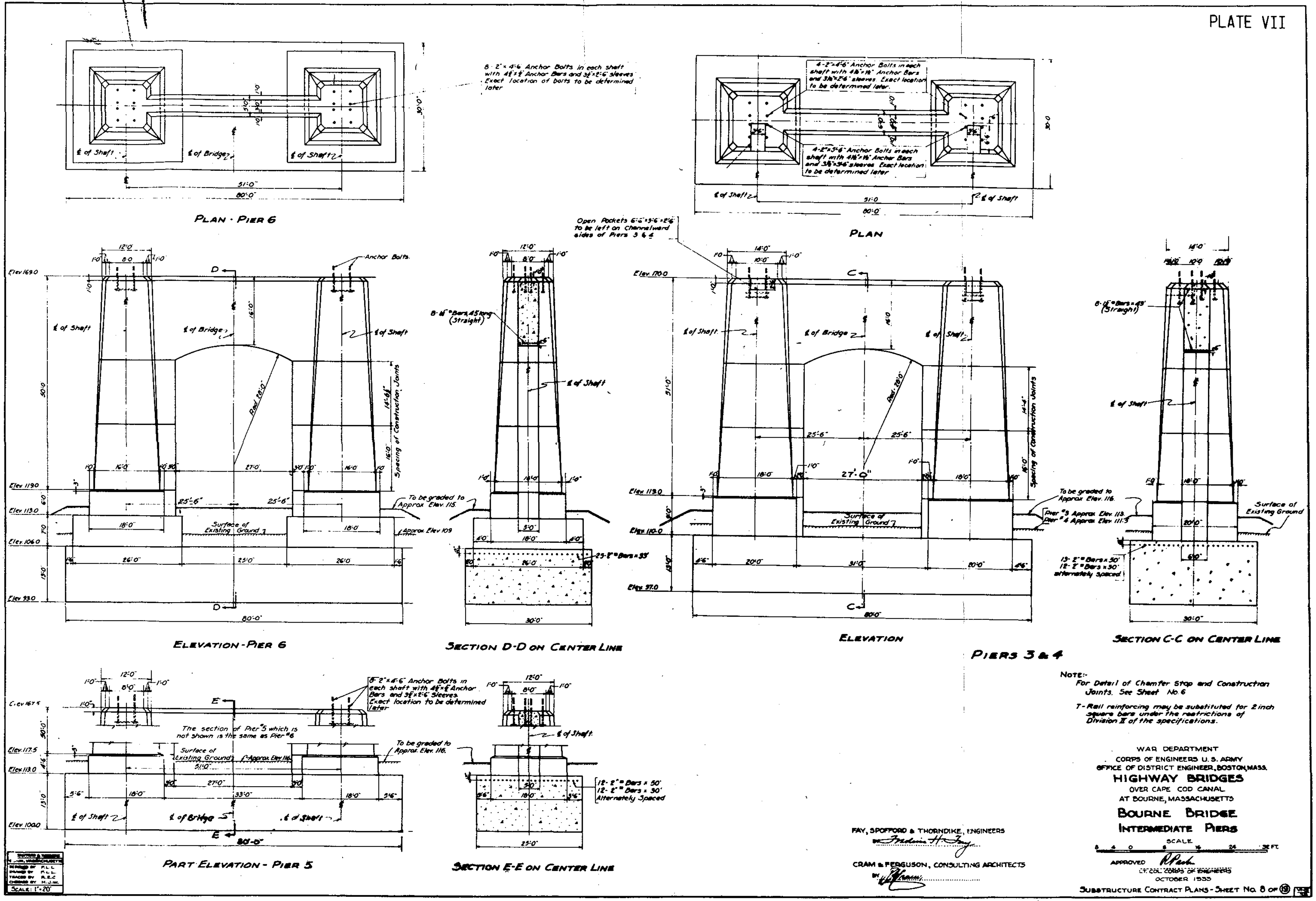
C-7


FRONT ELEVATION
END ELEVATION
SECTION A-A
SECTION B-B
PLAN
DETAIL OF CHAMFER STOP
DETAIL OF CONSTRUCTION JOINT
Steel anchorage at top of each shaft not shown in these elevations
Full lines indicate minimum base required if built by open cofferdam method.
Dash lines indicate minimum base required if built by caisson method.
If construction joint is used 11 T-rails 30' long and weighing at least 60 lbs per lin yard will be required as shown.
Steel rails required if construction joint is used
Granite facing
Legend:-
Concrete in section shown thus
Granite in section shown thus
Cast stone in section shown thus
Reinforcing steel shown thus
Steel T-rails shown thus
Note:-
For details of excavation and grading see Sheet No 2
The four Main Channel Piers (Piers 1 & 2) are identical at the Sagamore and Bourne Bridges for all four piers except for the slight variation in elevations and heights indicated.
Note:
Structural Steel Tower Reinforcement used in place of reinforcing bars.
WAR DEPARTMENT
CORPS OF ENGINEERS U. S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
SAGAMORE AND BOURNE BRIDGES
MAIN CHANNEL PIERS
SCALE
APPROVED
LT. COL. CORPS OF ENGINEERS
FAY, SPOFFORD & THORNDIKE, ENGINEERS
CRAM & FERGUSON, CONSULTING ARCHITECTS
SUBSTRUCTURE CONTRACT PLANS - SHEET No. 6 OF (19)
SCALE 1"=20'

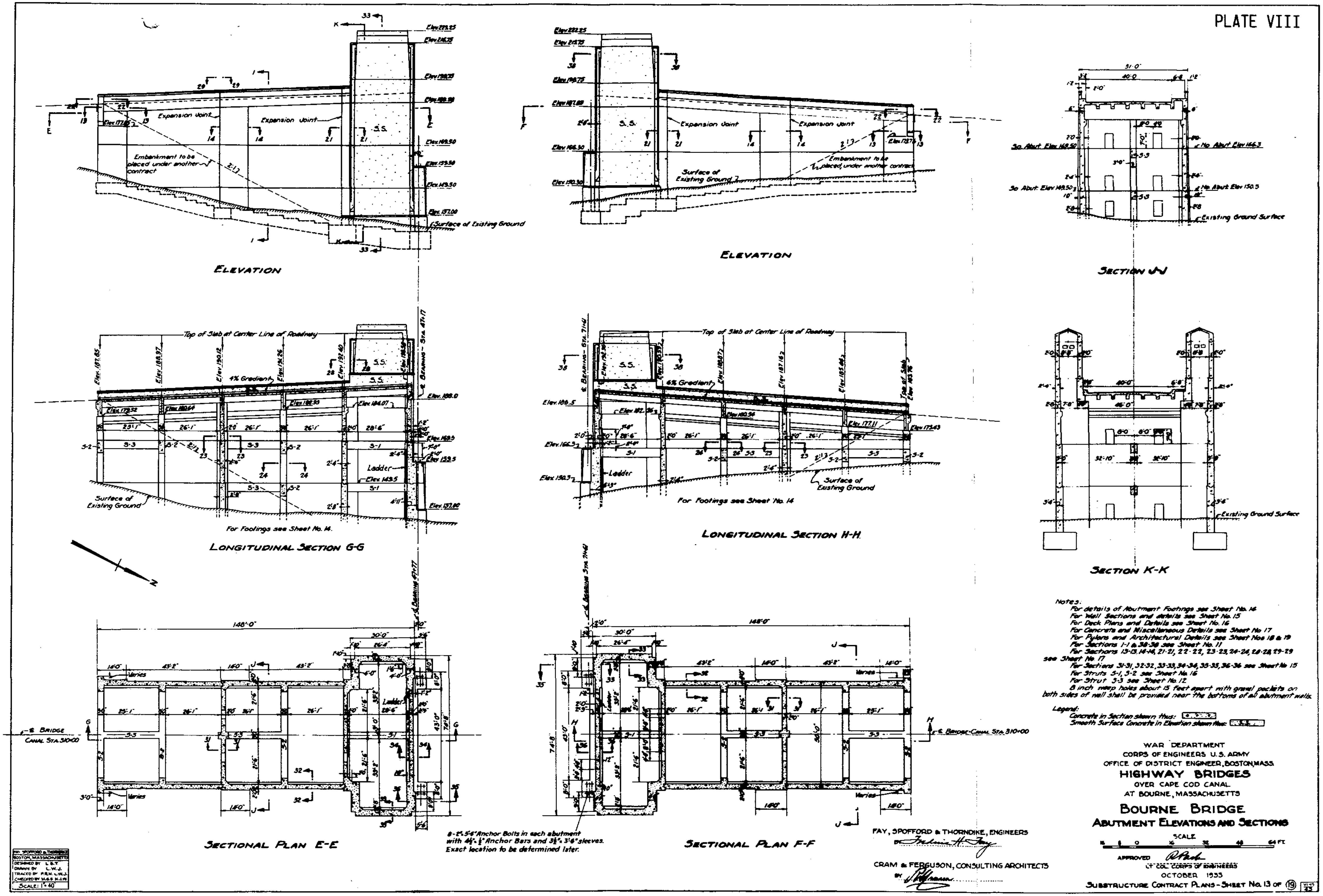
VL-56
C 8

PLATE VII



VL-56

PLATE VIII



Notes:
For details of Abutment Footings see Sheet No. 14
For Wall Sections and details see Sheet No. 15
For Deck Plans and Details see Sheet No. 16
For Concrete and Miscellaneous Details see Sheet No 17
For Pylons and Architectural Details see Sheet Nos 18 & 19
For Sections 1-1 & 38-38 see Sheet No. 11
For Sections 13-13, 14-14, 21-21, 22-22, 23-23, 24-24, 28-28, 29-29 see Sheet No. 17
For Sections 31-31, 32-32, 33-33, 34-34, 35-35, 36-36 see Sheet No 15
For Struts S-1, S-2 see Sheet No. 16
For Strut S-3 see Sheet No. 12
8 inch weep holes about 15 feet apart with gravel pockets on both sides of wall shall be provided near the bottoms of all abutment walls.

Legend:
Concrete in Section shown thus:
Smooth Surface Concrete in Elevation shown thus:

FAY, SPOFFORD & THORNDIKE, ENGINEERS
BY Frederic H. Fay

CRAM & FERGUSON, CONSULTING ARCHITECTS
BY

WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
BOURNE BRIDGE
ABUTMENT ELEVATIONS AND SECTIONS
SCALE
16 8 0 16 32 48 64 FT.
APPROVED
LT. COL. CORPS OF ENGINEERS
OCTOBER 1933
SUBSTRUCTURE CONTRACT PLANS - SHEET No. 13 OF 19

FAY, SPOFFORD & THORNDIKE
BOSTON, MASSACHUSETTS
DESIGNED BY L.S.T.
DRAWN BY L.W.J.
TRACED BY F.R.M. L.W.J.
CHECKED BY M.G.B. H.A.W.
SCALE: 1" = 40'

VL-56
C 10

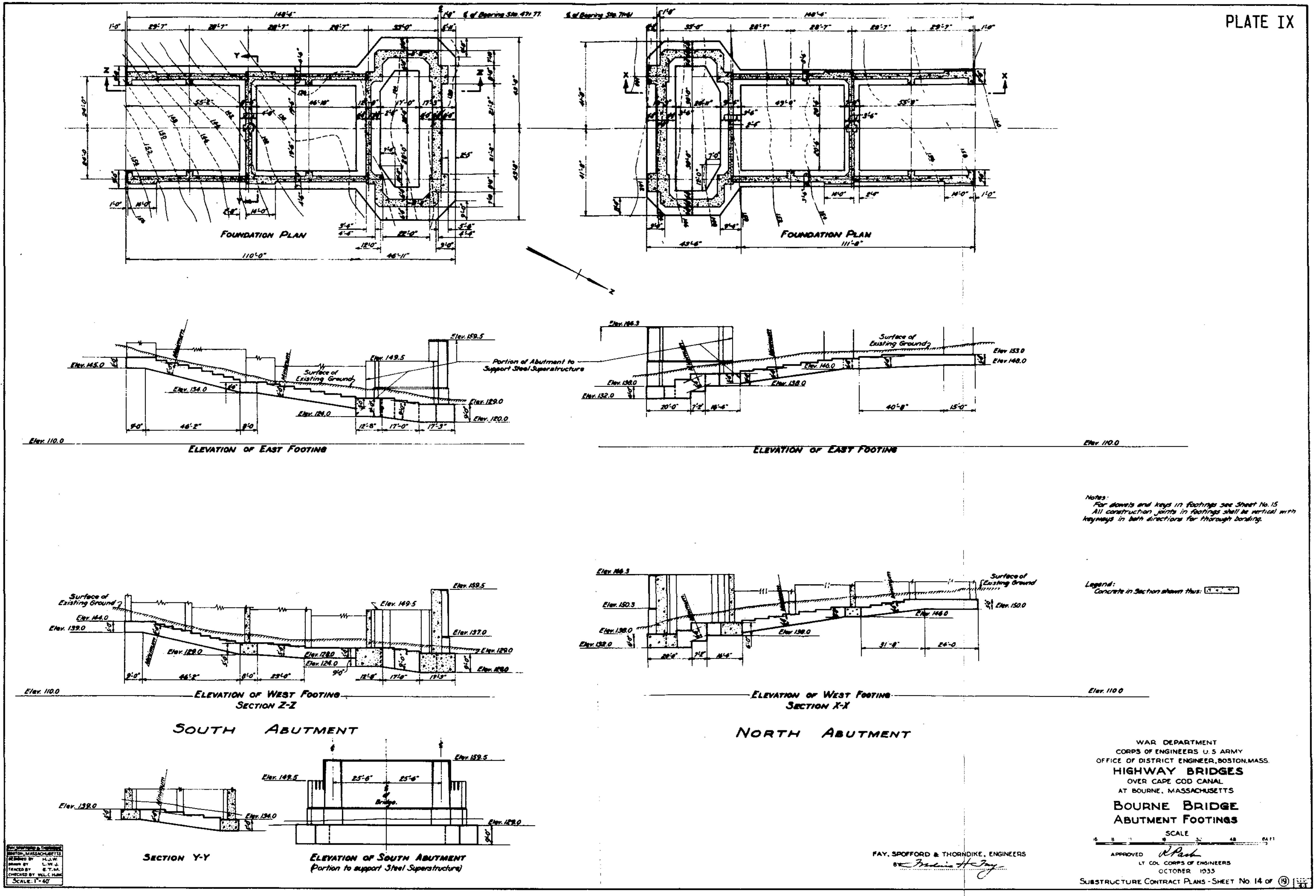
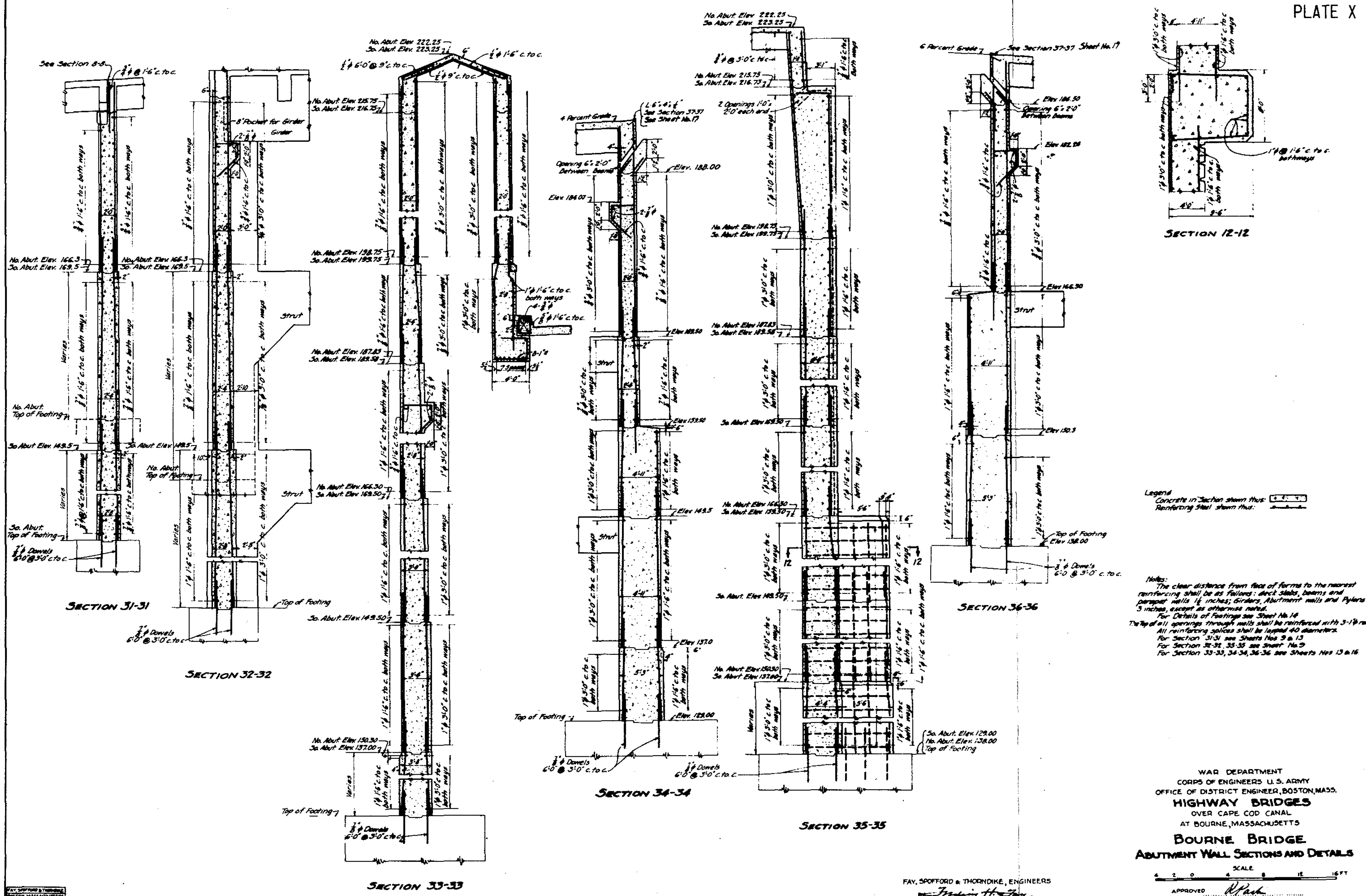


PLATE IX
FOUNDATION PLAN
FOUNDATION PLAN
ELEVATION OF EAST FOOTING
ELEVATION OF EAST FOOTING
Portion of Abutment to Support Steel Superstructure
Surface of Existing Ground
ELEVATION OF WEST FOOTING SECTION Z-Z
ELEVATION OF WEST FOOTING SECTION X-X
SOUTH ABUTMENT
NORTH ABUTMENT
SECTION Y-Y
ELEVATION OF SOUTH ABUTMENT (Portion to support Steel Superstructure)
Notes:
For dowels and keys in footings see Sheet No. 15
All construction joints in footings shall be vertical with keyways in both directions for thorough bonding.
Legend:
Concrete in Section shown thus:
WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
BOURNE BRIDGE
ABUTMENT FOOTINGS
SCALE
APPROVED
LT. COL. CORPS OF ENGINEERS
OCTOBER 1933
FAY, SPOFFORD & THORNDIKE, ENGINEERS
SUBSTRUCTURE CONTRACT PLANS - SHEET No. 14 OF 19
SCALE: 1" = 40'

PLATE X



Notes:
The clear distance from face of forms to the nearest reinforcing shall be as follows: deck slabs, beams and parapet walls 1½ inches; Girders, Abutment walls and Pylons 3 inches, except as otherwise noted.
For Details of Footings see Sheet No. 14
The top of all openings through walls shall be reinforced with 3-1¼ rods.
All reinforcing splices shall be lapped 40 diameters.
For Section 31-31 see Sheets Nos 9 & 13
For Section 32-32, 35-35 see Sheet No. 9
For Section 33-33, 34-34, 36-36 see Sheets Nos 13 & 16.

WAR DEPARTMENT
CORPS OF ENGINEERS U. S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.

HIGHWAY BRIDGES

OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS

BOURNE BRIDGE

ABUTMENT WALL SECTIONS AND DETAILS

SCALE
4 2 0 4 8 12 16 FT

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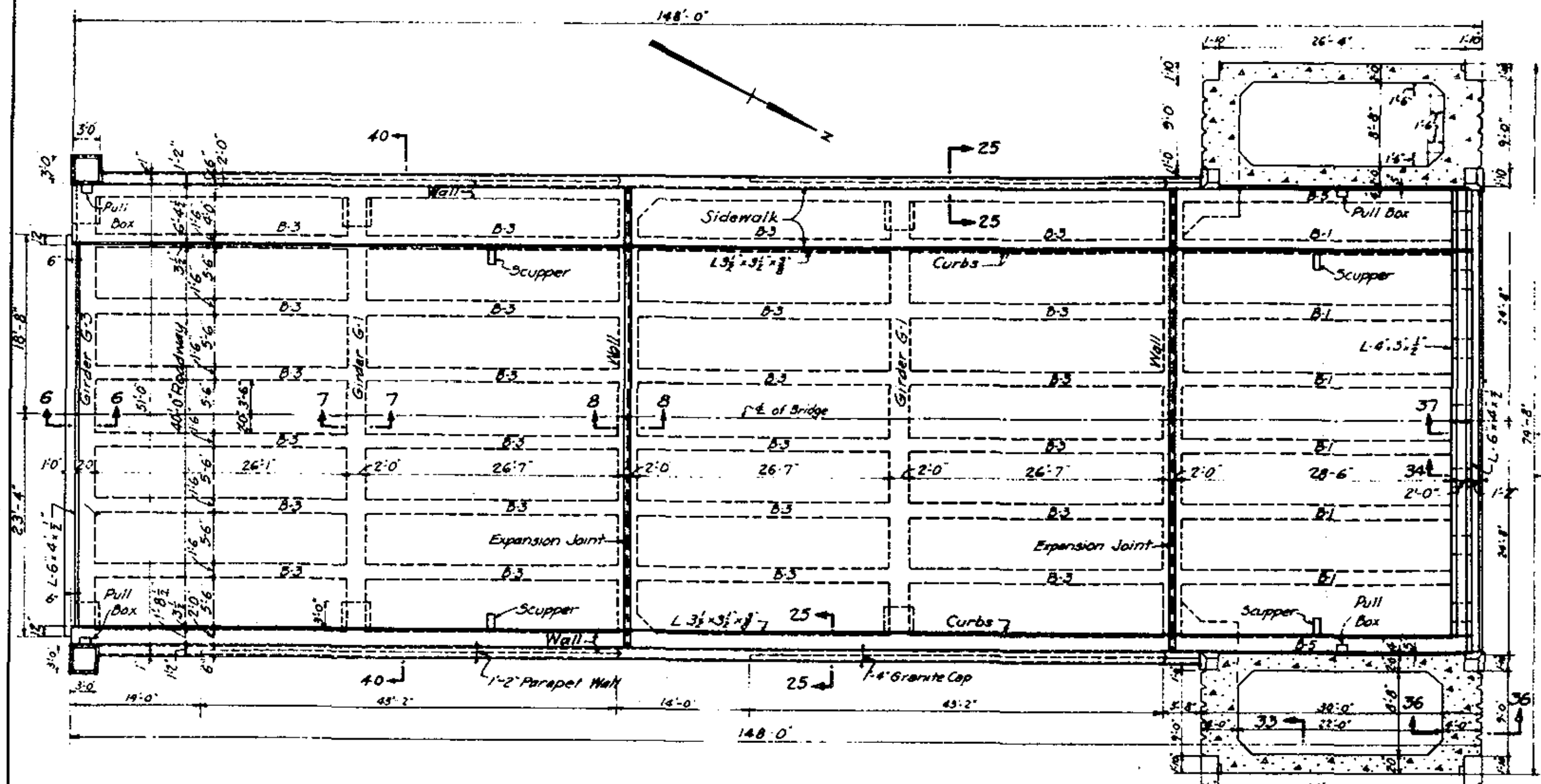
LT COL CORPS OF ENGINEERS
OCTOBER 1933

FAY, SPOFFORD & THORNDIKE, ENGINEERS

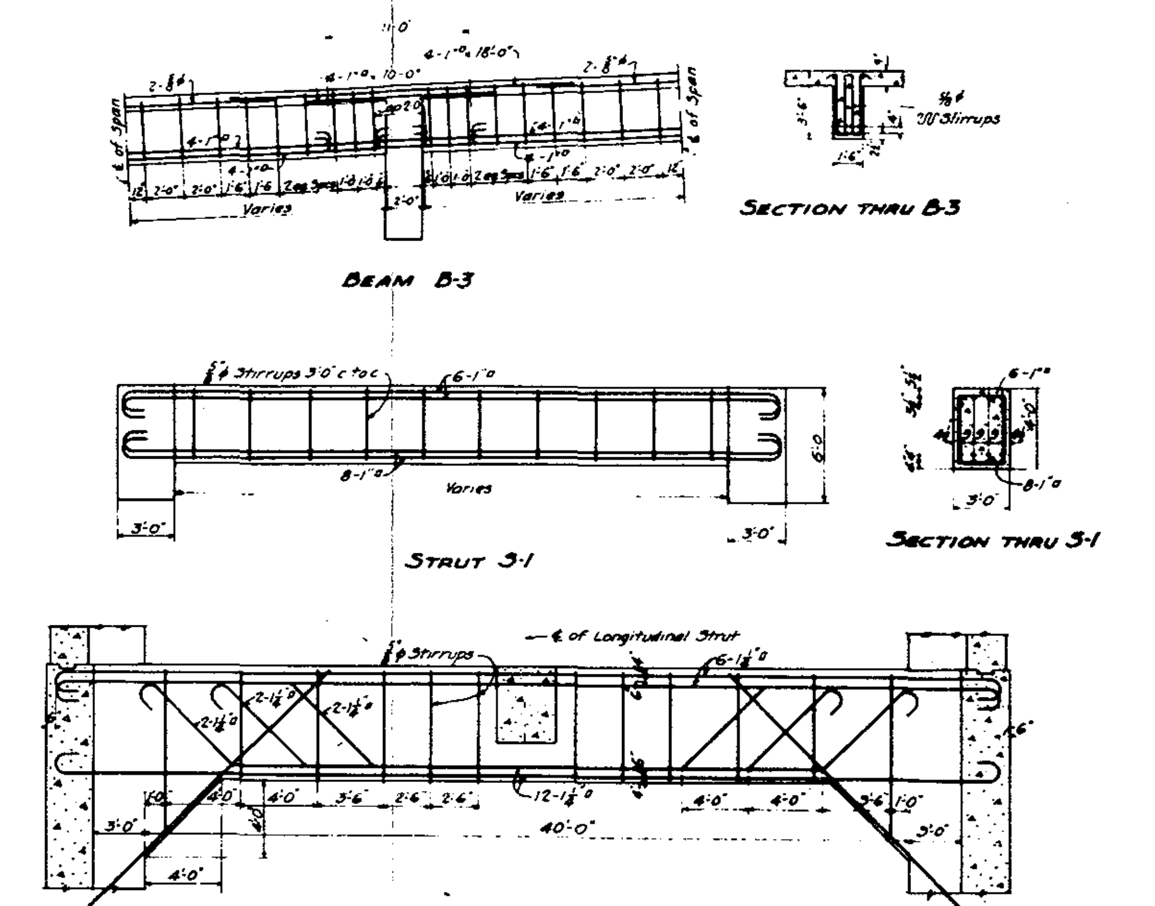
SUBSTRUCTURE CONTRACT PLANS - SHEET No. 15 OF 19

VL-56
C 12

PLATE XI



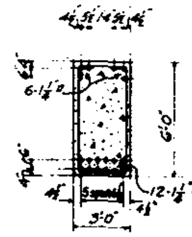
SOUTH ABUTMENT DECK PLAN
(North Abutment similar but opposite hand)



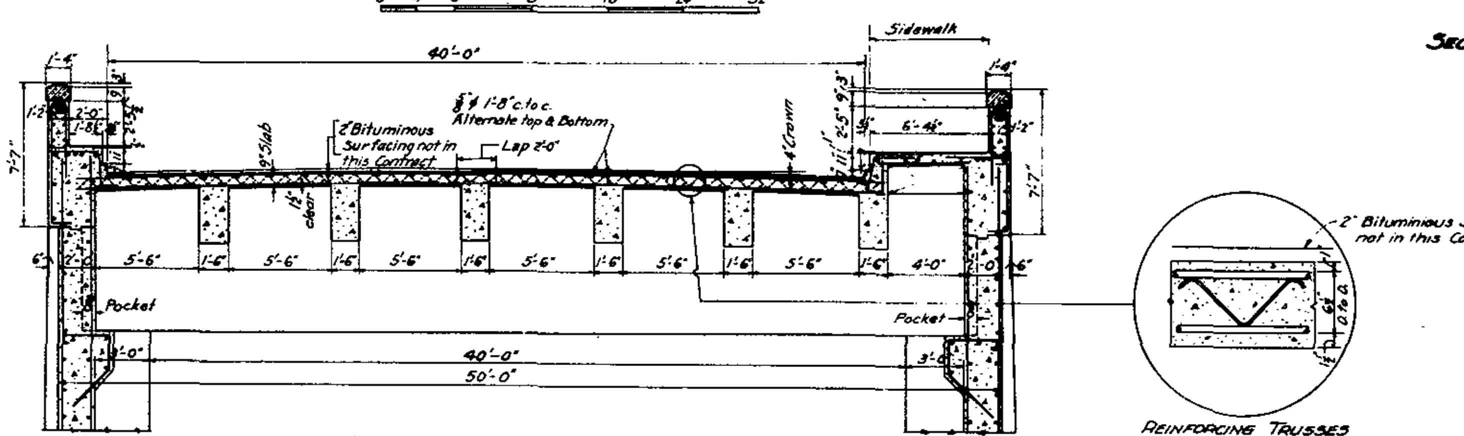
BEAM B-3

STRUT S-1

STRUT S-2

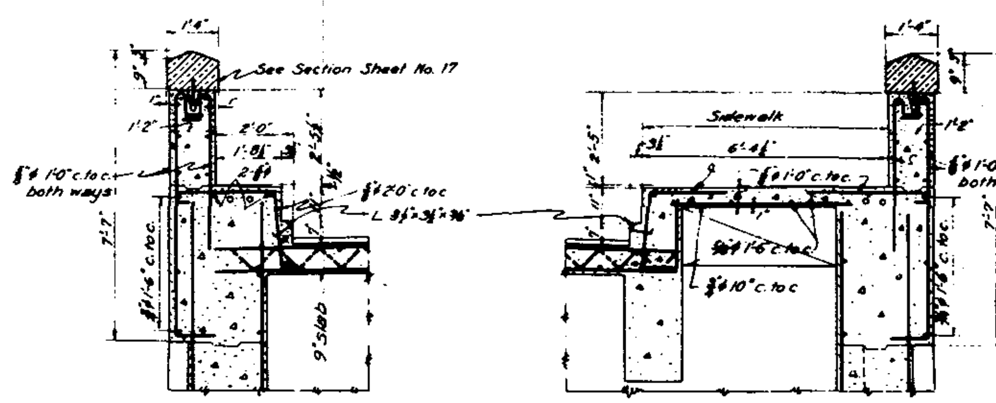


SECTION THRU S-2

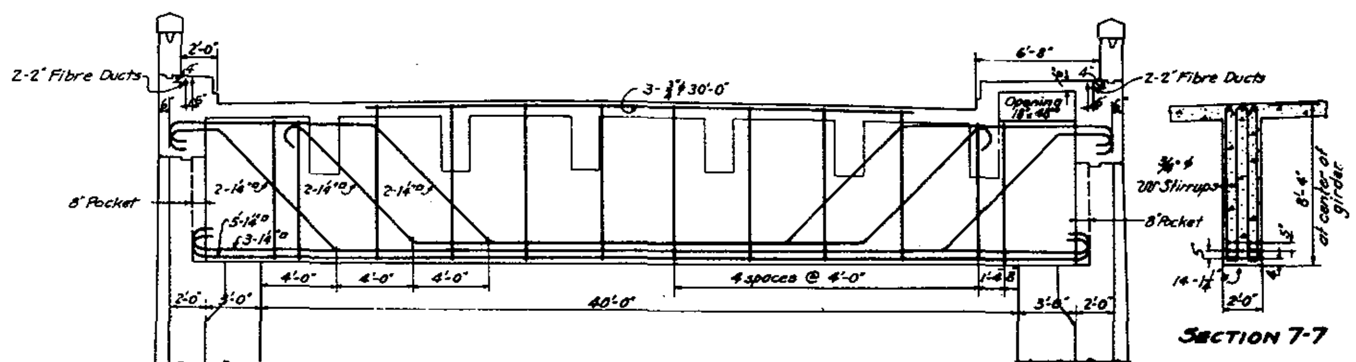


CROSS SECTION OF DECK
SECTION 40-40

REINFORCING TRUSSES
Spaced 6" c. to c. - depth 6½" out to out of truss and cambered as shown.
Top and Bottom chord - Area of each chord = 0.44 Sq. In.
Web area to average ¼ pound per linear foot of truss.



DETAIL OF PARAPET AND SIDEWALK
Scale ½" = 1'-0"



GIRDER G-1
(For Girder G-3 see Section 6-6 Sheet No. 17
For Girder G-2 see Section 9-9 Sheet No. 17)

SECTION 7-7

Legend
Concrete in Section shown thus:
Reinforcing Steel shown thus:

Notes:
For Sections 6-6, 8-8, 25-25, 37-37 see Sheet No. 17
For Sections 33-33, 34-34, 36-36 see Sheet No. 15
For Beam B-1 see Sheet No. 12
For Beam B-5 see Sheet No. 15

WAR DEPARTMENT
CORPS OF ENGINEERS U. S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
BOURNE BRIDGE
ABUTMENT DECK PLANS AND DETAILS
SCALE EXCEPT AS SHOWN

FAY, SPOFFORD & THORNDIKE, ENGINEERS
BY Frederic H. Fay

APPROVED
LT. COL. CORPS OF ENGINEERS
OCTOBER 1933

SUBSTRUCTURE CONTRACT PLANS - SHEET No. 16 OF 19

FAY, SPOFFORD & THORNDIKE
BOSTON, MASSACHUSETTS
DESIGNED BY L.B.T.
DRAWN BY H.L.K.
TRACED BY T.V.A. W.H.K.
CHECKED BY H.B.S. H.A.W.
SCALE: 1"=10' EXCEPT AS SHOWN

VL-56

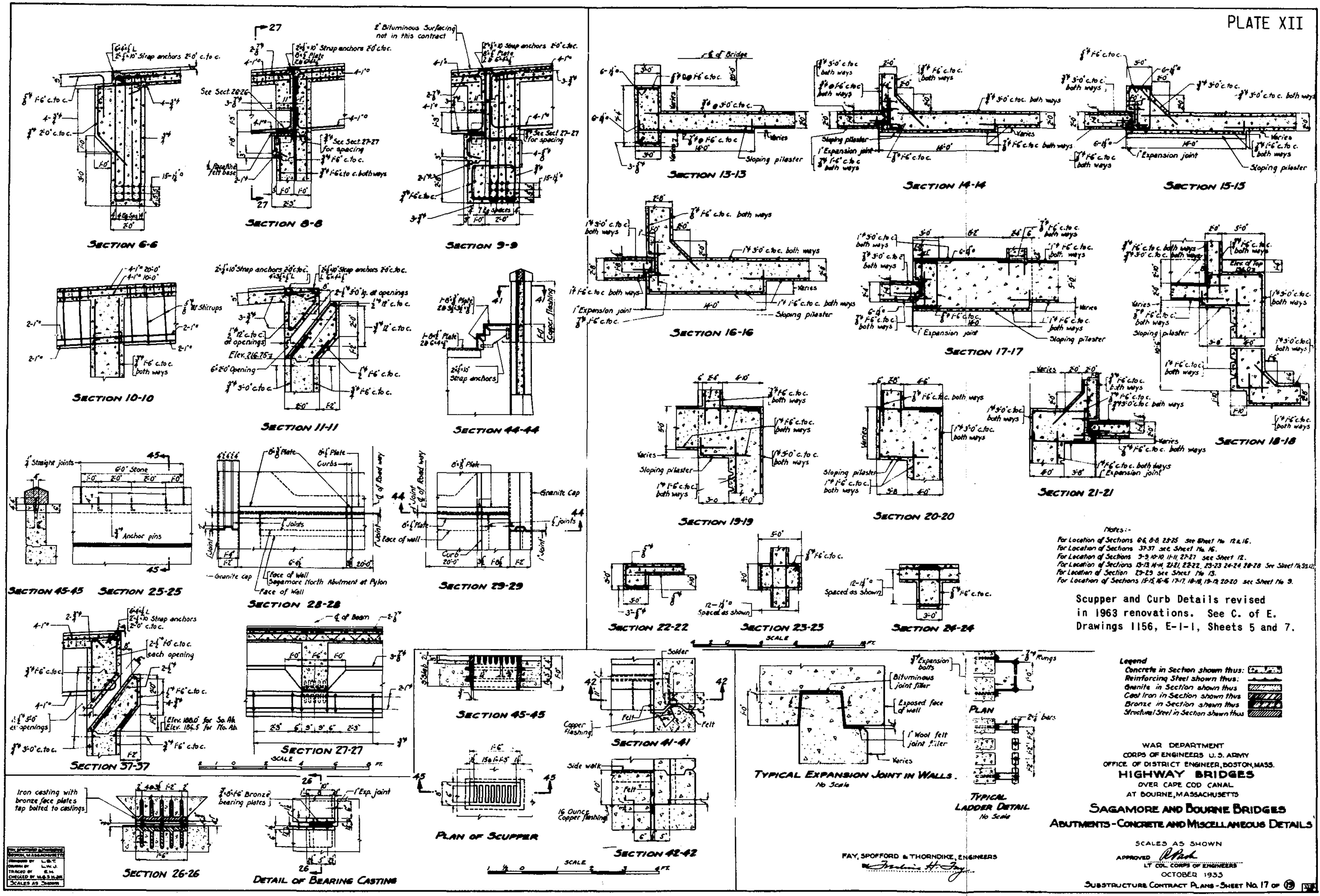


PLATE XII
SECTION 6-6
SECTION 8-8
SECTION 9-9
SECTION 10-10
SECTION 11-11
SECTION 44-44
SECTION 13-13
SECTION 14-14
SECTION 15-15
SECTION 16-16
SECTION 17-17
SECTION 18-18
SECTION 19-19
SECTION 20-20
SECTION 21-21
SECTION 22-22
SECTION 23-23
SECTION 24-24
SECTION 45-45
SECTION 25-25
SECTION 28-28
SECTION 29-29
SECTION 37-37
SECTION 27-27
SECTION 45-45
SECTION 41-41
SECTION 42-42
SECTION 26-26
DETAIL OF BEARING CASTING
PLAN OF SCUPPER
TYPICAL EXPANSION JOINT IN WALLS.
No Scale
TYPICAL LADDER DETAIL
No Scale
Notes:-
For Location of Sections 6-6, 8-8, 25-25 see Sheet No. 12 & 16.
For Location of Sections 37-37 see Sheet No. 16.
For Location of Sections 9-9, 10-10, 11-11, 27-27 see Sheet 12.
For Location of Sections 13-13, 14-14, 21-21, 22-22, 23-23, 24-24, 28-28 See Sheet No. 15.
For Location of Section 29-29 see Sheet No. 13.
For Location of Sections 15-15, 16-16, 17-17, 18-18, 19-19, 20-20 see Sheet No. 9.
Scupper and Curb Details revised in 1963 renovations. See C. of E. Drawings 1156, E-1-1, Sheets 5 and 7.
Legend
Concrete in Section shown thus:
Reinforcing Steel shown thus:
Granite in Section shown thus
Cast Iron in Section shown thus
Bronze in Section shown thus
Structural Steel in Section shown thus
WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
SAGAMORE AND BOURNE BRIDGES
ABUTMENTS - CONCRETE AND MISCELLANEOUS DETAILS
SCALES AS SHOWN
APPROVED
LT. COL. CORPS OF ENGINEERS
OCTOBER 1933
SUBSTRUCTURE CONTRACT PLANS - SHEET NO. 17 OF 19
FAY, SPOFFORD & THORNDIKE, ENGINEERS

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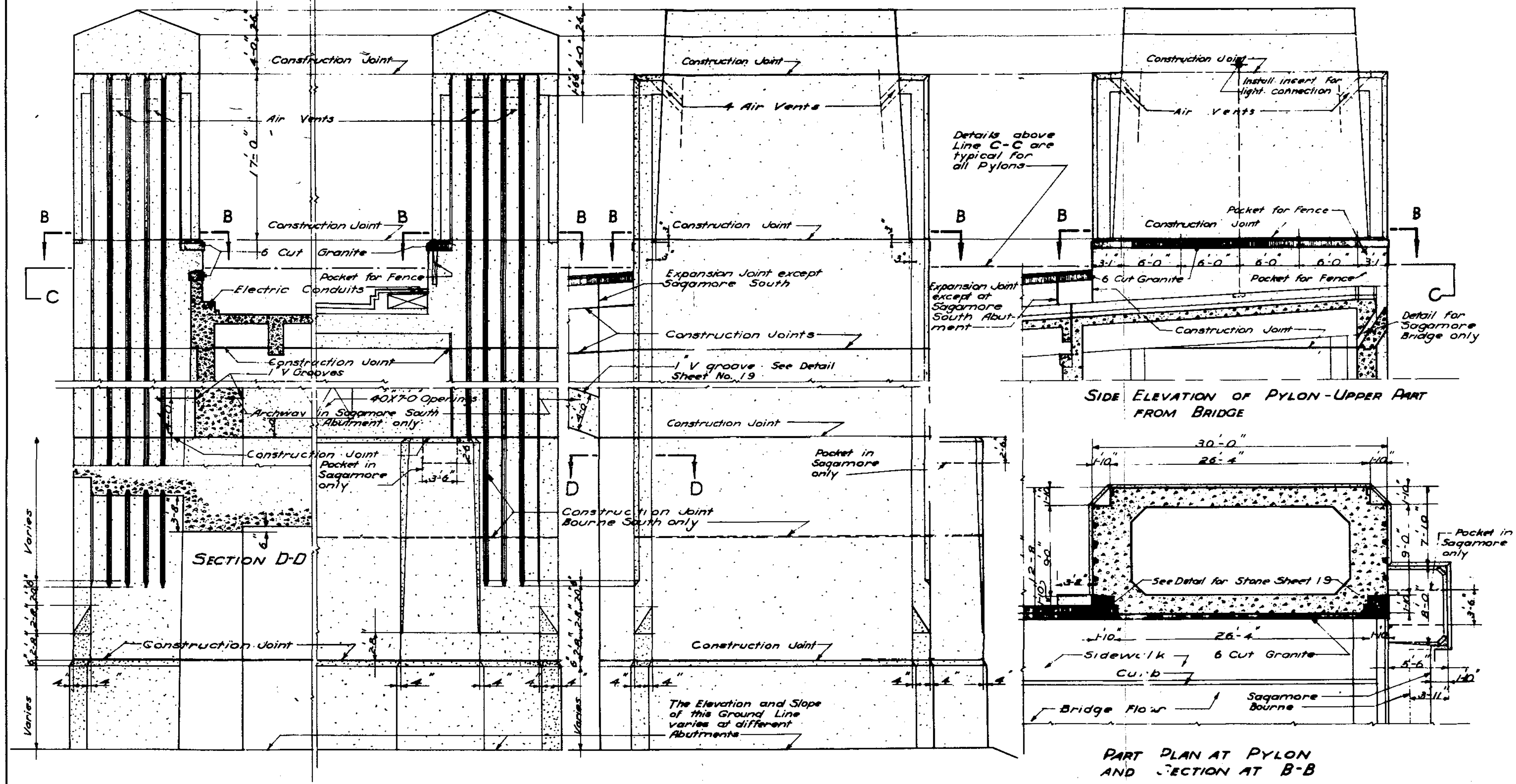
PLATE XIII



END ELEVATION OF PYLON FROM APPROACH SIDE

END ELEVATION OF PYLON FROM TRUSS SIDE

SIDE ELEVATION OF PYLON FROM OUTSIDE OF BRIDGE

PART PLAN AT PYLON AND SECTION AT B-B

FAY, SPOFFORD & THORNDIKE, ENGINEERS
BY Frederic H. Fay

CRAM & FERGUSON, CONSULTING ARCHITECTS
BY

WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS

SAGAMORE AND BOURNE BRIDGES
ABUTMENTS-PYLONS

SCALE EXCEPT AS SHOWN

APPROVED R. Park
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OCTOBER 1933

SUBSTRUCTURE CONTRACT PLANS - SHEET No. 18 OF 19

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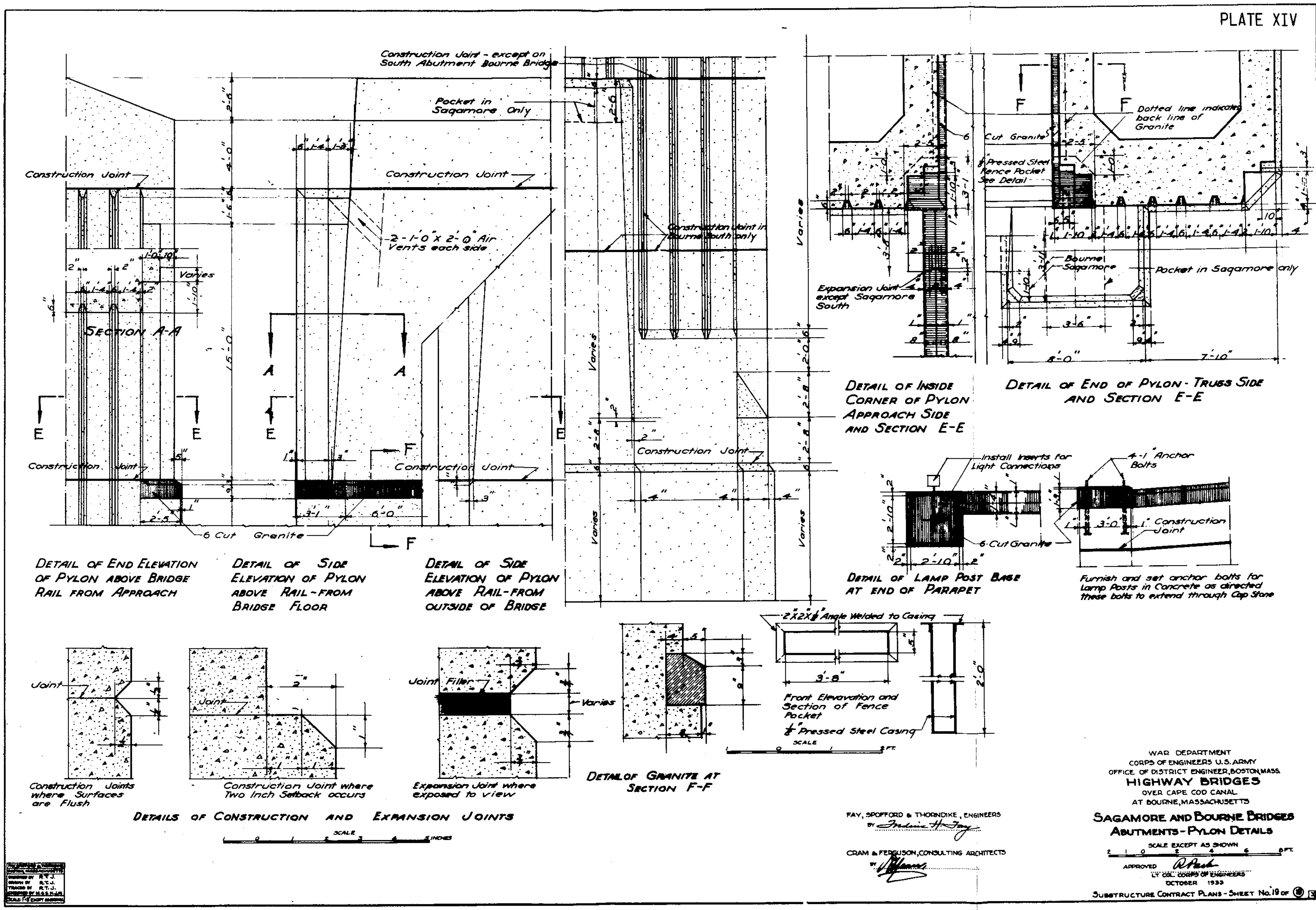
PLATE XIV
Construction Joint - except on South Abutment Bourne Bridge
Pocket in Sagamore Only
Construction Joint
2-1'-0" X 2'-0" Air Vents each side
Construction Joint in Bourne South only
Section A-A
Varies
Construction Joint
6 Cut Granite
Detail of End Elevation of Pylon above Bridge Rail from Approach
Detail of Side Elevation of Pylon above Rail - from Bridge Floor
Detail of Side Elevation of Pylon above Rail - from outside of Bridge
Cut Granite
Dotted line indicates back line of Granite
1/8" Pressed Steel Fence Pocket See Detail
Expansion Joint except Sagamore South
Bourne
Sagamore
Pocket in Sagamore only
Detail of Inside Corner of Pylon Approach Side and Section E-E
Detail of End of Pylon - Truss Side and Section E-E
Install Inserts for Light Connections
4-1" Anchor Bolts
6-Cut Granite
Construction Joint
Detail of Lamp Post Base at End of Parapet
Furnish and set anchor bolts for Lamp Posts in Concrete as directed these bolts to extend through Cap Stone
Joint
Joint Filler
Varies
Construction Joints where Surfaces are Flush
Construction Joint where Two Inch Setback occurs
Expansion Joint where exposed to view
Details of Construction and Expansion Joints
Detail of Granite at Section F-F
2"X2"X1/4" Angle Welded to Casing
Front Elevation and Section of Fence Pocket
1/8" Pressed Steel Casing
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OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
SAGAMORE AND BOURNE BRIDGES
ABUTMENTS - PYLON DETAILS
SCALE EXCEPT AS SHOWN
APPROVED
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CRAM & FERGUSON, CONSULTING ARCHITECTS
SUBSTRUCTURE CONTRACT PLANS - SHEET No. 19

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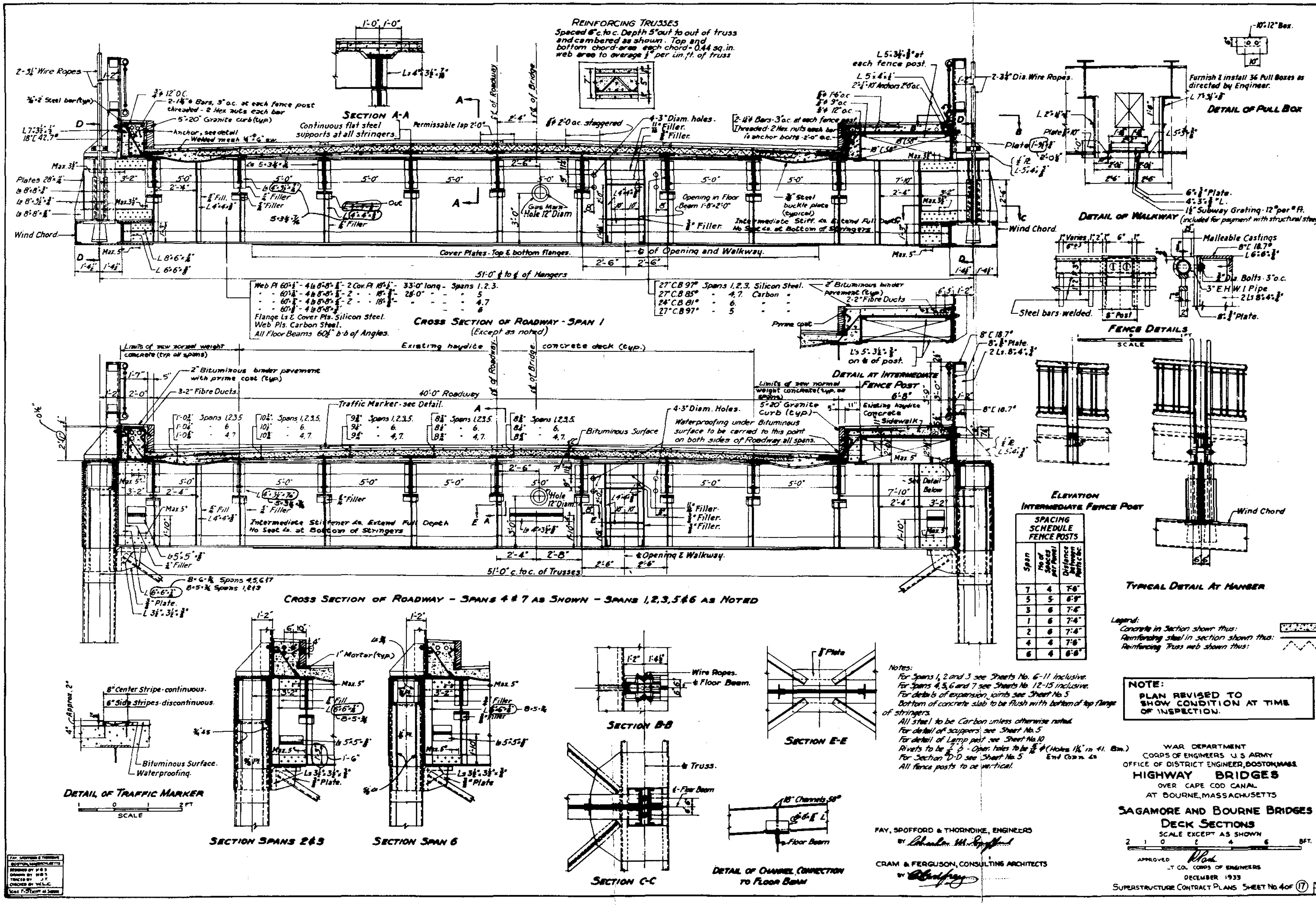

SPACING SCHEDULE FENCE POSTS

Span	No. of Spaces per Panel	Distance Between Posts c. to c.
7	4	7'-6"
5	5	6'-9"
3	6	7'-6"
1	6	7'-4"
2	6	7'-4"
4	4	7'-6"
6	4	6'-6"

Notes:
For Spans 1, 2 and 3 see Sheets No. 6-11 inclusive.
For Spans 4, 5, 6 and 7 see Sheets No. 12-15 inclusive.
For details of expansion joints see Sheet No. 5
Bottom of concrete slab to be flush with bottom of top flange of stringers.
All steel to be Carbon unless otherwise noted.
For detail of scuppers see Sheet No. 5
For detail of Lamp post see Sheet No. 10
Rivets to be ⅞" ϕ - Open holes to be 15/16" ϕ (Holes 1⅛" in Fl. Bm. End Conn. Ls)
For Section D-D see Sheet No. 5
All fence posts to be vertical.

NOTE:
PLAN REVISED TO SHOW CONDITION AT TIME OF INSPECTION.

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OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
SAGAMORE AND BOURNE BRIDGES
DECK SECTIONS
SCALE EXCEPT AS SHOWN

FAY, SPOFFORD & THORNDIKE, ENGINEERS

CRAM & FERGUSON, CONSULTING ARCHITECTS

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DECEMBER 1933
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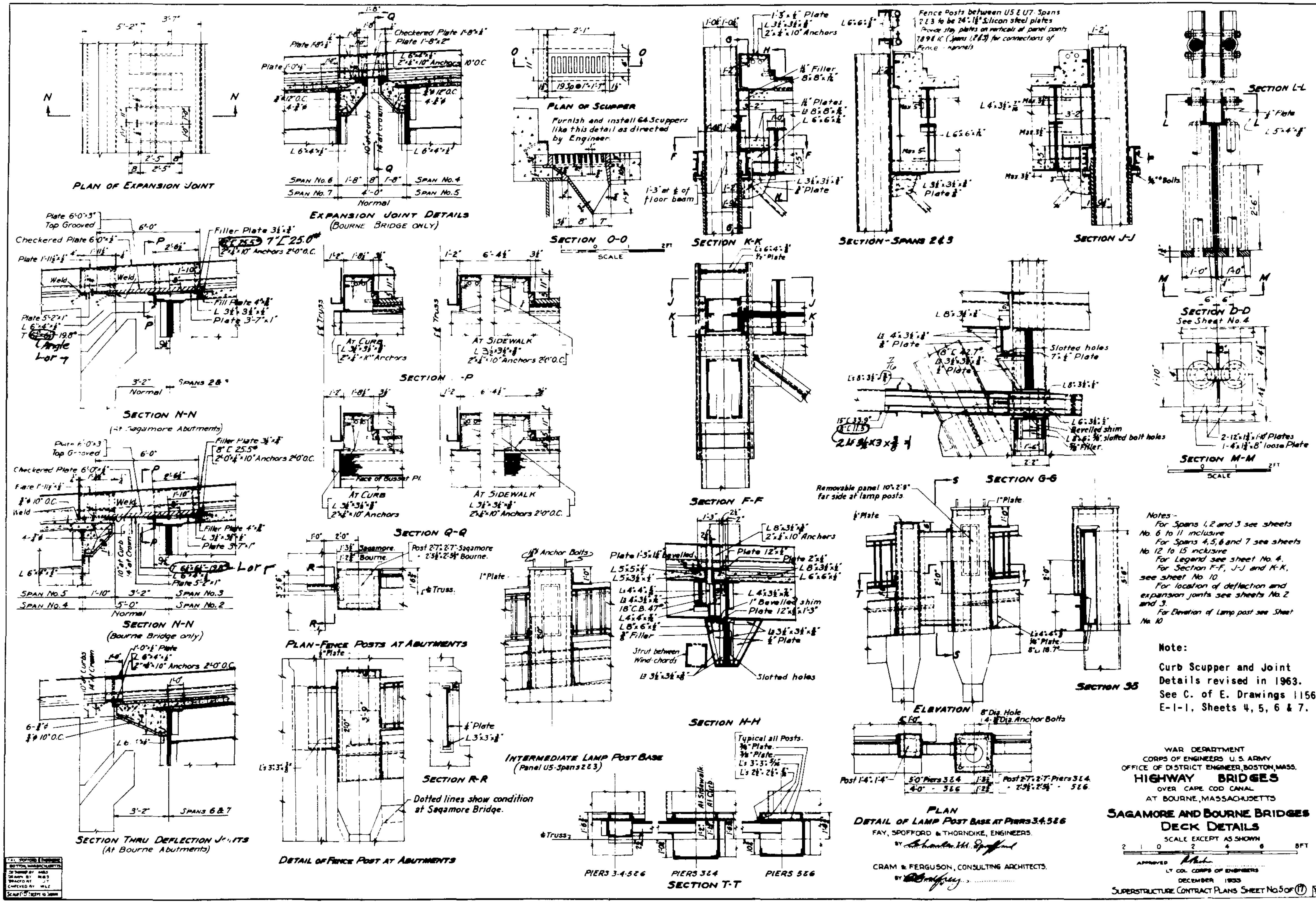
C 11


PLAN OF EXPANSION JOINT
EXPANSION JOINT DETAILS
(BOURNE BRIDGE ONLY)
PLAN OF SCUPPER
Furnish and install 64 Scuppers like this detail as directed by Engineer.
SECTION O-O
SCALE
SECTION K-K
SECTION-SPANS 2 & 3
SECTION J-J
SECTION L-L
Fence Posts between U5 & U7 Spans 2 & 3 to be 24" x 1 1/8" Silicon steel plates
SECTION N-N
(At Sagamore Abutments)
SECTION P-P
AT CURB
AT SIDEWALK
SECTION F-F
SECTION D-D
See Sheet No. 4
SECTION M-M
SECTION N-N
(Bourne Bridge only)
SECTION Q-Q
PLAN-FENCE POSTS AT ABUTMENTS
SECTION G-G
SECTION H-H
INTERMEDIATE LAMP POST BASE
(Panel U5-Spans 2 & 3)
SECTION R-R
Dotted lines show condition at Sagamore Bridge.
SECTION THRU DEFLECTION JOINTS
(At Bourne Abutments)
DETAIL OF FENCE POST AT ABUTMENTS
PIERS 3-4-5 & 6
PIERS 3 & 4
PIERS 5 & 6
SECTION T-T
ELEVATION
SECTION SS
PLAN
DETAIL OF LAMP POST BASE AT PIERS 3, 4, 5 & 6
Notes -
For Spans 1, 2 and 3 see sheets No. 6 to 11 inclusive
For Spans 4, 5, 6 and 7 see sheets No. 12 to 15 inclusive
For Legend see sheet No. 4.
For Section F-F, J-J and K-K, see sheet No. 10
For location of deflection and expansion joints see sheets No. 2 and 3
For Elevation of Lamp post see Sheet No. 10
Note:
Curb Scupper and Joint Details revised in 1963. See C. of E. Drawings 1156, E-1-1, Sheets 4, 5, 6 & 7.
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CRAM & FERGUSON, CONSULTING ARCHITECTS.
WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
SAGAMORE AND BOURNE BRIDGES
DECK DETAILS
SCALE EXCEPT AS SHOWN
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DECEMBER 1933
SUPERSTRUCTURE CONTRACT PLANS SHEET No. 5 OF 17

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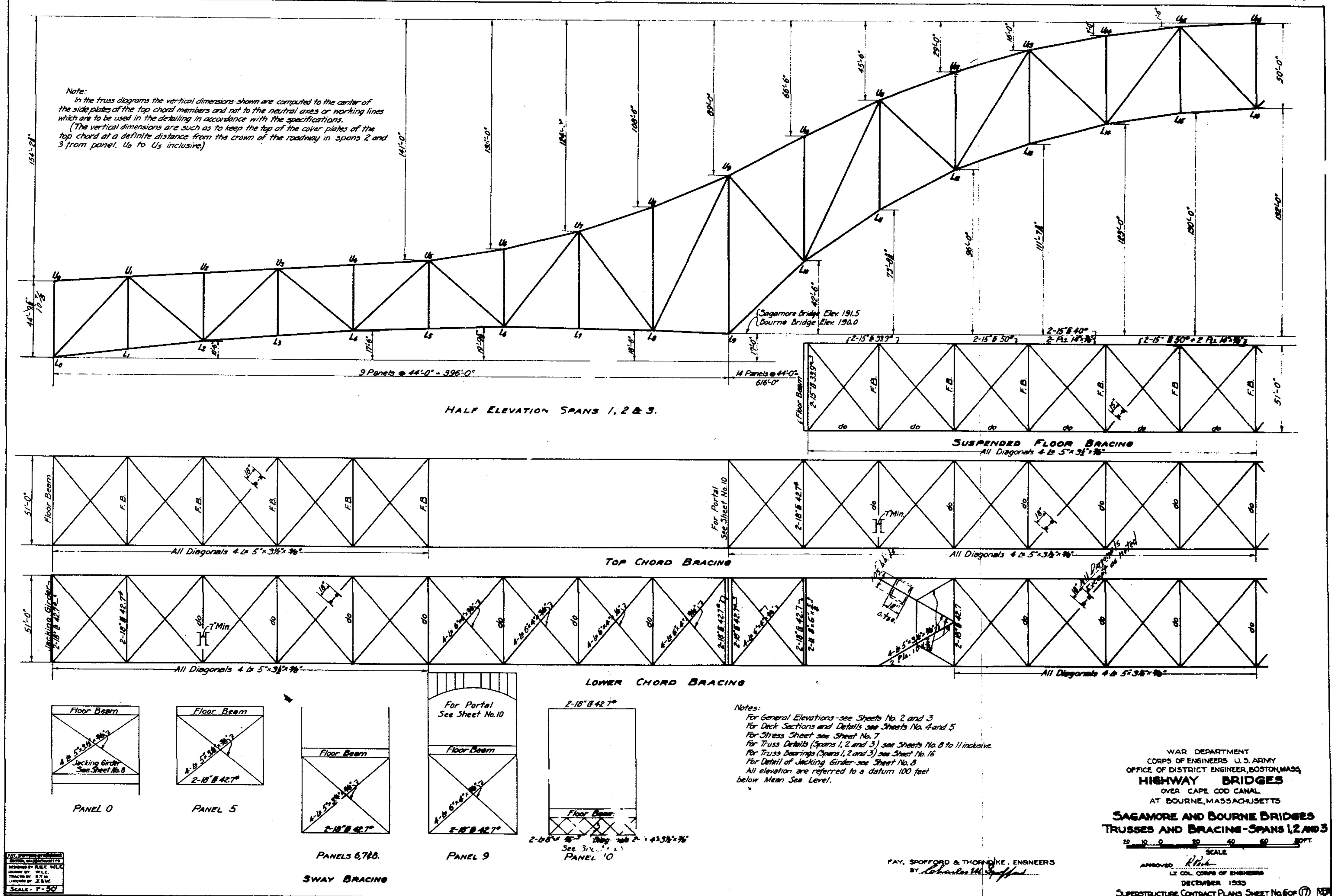
Note:
In the truss diagrams the vertical dimensions shown are computed to the center of the side plates of the top chord members and not to the neutral axes or working lines which are to be used in the detailing in accordance with the specifications.
(The vertical dimensions are such as to keep the top of the cover plates of the top chord at a definite distance from the crown of the roadway in spans 2 and 3 from panel U_0 to U_5 inclusive.)
Sagamore Bridge Elev. 191.5
Bourne Bridge Elev. 190.0
9 Panels @ 44'-0" = 396'-0"
14 Panels @ 44'-0" = 616'-0"
HALF ELEVATION SPANS 1, 2 & 3.
SUSPENDED FLOOR BRACING
All Diagonals 4 Ls 5"x3½"x⅜"
TOP CHORD BRACING
For Portal See Sheet No. 10
LOWER CHORD BRACING
Jacking Girder
Floor Beam
PANEL 0
PANEL 5
PANELS 6,7 & 8.
PANEL 9
PANEL 10
SWAY BRACING
Notes:
For General Elevations - see Sheets No. 2 and 3
For Deck Sections and Details see Sheets No. 4 and 5
For Stress Sheet see Sheet No. 7
For Truss Details (Spans 1, 2 and 3) see Sheets No. 8 to 11 inclusive.
For Truss Bearings (Spans 1, 2 and 3) see Sheet No. 16
For Detail of Jacking Girder - see Sheet No. 8
All elevation are referred to a datum 100 feet below Mean Sea Level.
FAY, SPOFFORD & THORNDIKE, ENGINEERS
WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
SAGAMORE AND BOURNE BRIDGES
TRUSSES AND BRACING - SPANS 1, 2 AND 3
SCALE
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DECEMBER 1933
SUPERSTRUCTURE CONTRACT PLANS SHEET No. 6 OF 17
SCALE - 1"-50'

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PLATE XVIII

Members in Spans 2 and 3

Bar	Dead Stress	Live Stress Unif.	Live Stress Conc.	Live Stress Total	% I	Imp.	Live & Imp.	Erection Dead + Trav. A	Erection Dead + Trav. B	Wind Comp. Struct.	Wind Erection A	Wind Erection B	D+L+I	D+2(L+I)+W	Total Erect A	Total Erect B	Design Stress Basis of 32,400	Length	Make-up of Sections		Assembly	Radius of Gyration	Area
L0L1	+411	+193 / -69	+30 / -5	+223 / -74	6 / 7	+13 / -5	+236 / -79	+1	+688	+80 / -152	+156 / -232	+120 / -216	+647	+965	+157 / -231	+808	+963 / -210	44.31	4 Ls 6"×4"×3/8" 2 Webs 30"×1/2"	(Si)	[ ]		G. 44.6 N. 36.1
L1L2	+411	+193 / -69	+30 / -5	+223 / -74	6 / 7	+13 / -5	+236 / -79	+1	+688	+139 / -217	+233 / -301	+216 / -301	+647	+1022	+234 / -300	+904	+1022 / -273	44.31	Same as L0L1	(Si)	do		G. 44.6 N. 36.1
L2L3	+646	+442 / -226	+64 / -17	+505 / -242	6 / 7	+28 / -16	+534 / -258	-312	+1512	+375 / -451	+631 / -699	+551 / -626	+1180	+2120 / -547	+319 / -1011	+2063	+2120 / -920	44.14	4 Ls 6"×4"×9/16" 4 Webs 30"×1/2"	(Si)	[ ]		G. 81.2 N. 66.
L3L4	+646	+442 / -226	+64 / -17	+505 / -242	6 / 7	+28 / -16	+534 / -258	-312	+1512	+430 / -500	+691 / -750	+625 / -688	+1180	+2175 / -596	+379 / -1062	+2137	+2175 / -967	44.14	Same as L2L3	(Si)	do		G. 81.2 N. 66.
L4L5	-129	+417 / -369	+56 / -27	+473 / -396	6 / 7	+26 / -26	+499 / -422	-1074	+1240	+620 / -670	+1028 / -1077	+892 / -942	+614 / -756	+1982 / -2104	-2151	+2132	+1982 / -2104	44.02	4 Ls 6"×4"×1/2" 4 Webs 30"×5/16"	(Si)	[ ]	rx = 9.7 ry = 11.3	G. 94. N. 76.6
L5L6	-129	+417 / -369	+56 / -27	+473 / -396	6 / 7	+26 / -26	+499 / -422	-1074	+1240	+624 / -676	+974 / -1077	+873 / -942	+614 / -756	+1986 / -2104	-2151	+2113	+1986 / -2104	44.02	Same as L4L5	(Si)	do	do	G. 94. N. 76.6
L6L7	-1340	+132 / -387	+22 / -27	+154 / -414	7 / 5	+11 / -22	+165 / -436	-1660	-37	+405 / -493	+531 / -660	+506 / -615	-1776	-2705	-2320	+469 / -652	+427 / -2705	44.00	4 Ls 6"×4"×9/16" 4 Webs 30"×5/16"	(Si)	[ ]	rx = 9.6 ry = 11.1	G. 111.2 N. 90.6
L7L8	-1340	+132 / -387	+22 / -27	+154 / -414	7 / 5	+11 / -22	+165 / -436	-1660	-37	+327 / -423	+374 / -538	+352 / -512	-1776	-2635	-2198	+315 / -549	+286 / -2635	44.00	4 Ls 6"×4"×9/16" 4 Webs 30"×9/16" 3/4	(Si)	[ ]	rx = 9.6 ry = 11.1	G. 103.8 N. 84.5
L8L9	-2070	+51 / -481	+6 / -22	+56 / -503	10 / 4	+5 / -22	+62 / -525	-1781	-1033	+305 / -355	+55 / -127	+28 / -144	-2595	-3475	-1908	-1177	-3475	44.03	4 Ls 8"×6"×9/16" 4 Webs 30"×3/4"	(Si)	[ ]	rx = 10.2 ry = 10.8	G. 132.8 N.
U0U1	0	0 / 0	0 / 0	0 / 0	0 / 0	0 / 0	0 / 0	0		-45	-69	-49	0 / 0	-45	-69	-49	-63	44.08	1-C.P. 24"×3/8" 23" 4-Ls 6"×4"×3/8" 2-Webs 30"×3/8"		[ ]	rx = 11.4 ry = 10.7	G. 45.9
U1U2	-651	+147 / -349	+11 / -53	+158 / -401	7 / 6	+10 / -23	+168 / -424	+96	-1225	+111 / -115	+335 / -426	+210 / -272	-1075	-1614	+431 / -330	-1497	+992 / -1614	44.08	1-C.P. 24"×1/2" 23" 4-Ls 6"×4"×3/8" 2-Webs 30"×3/4"	(Si)	[ ]	rx = 10.8 ry = 11.	G. 71.4 N. 59.5
U2U3	-651	+147 / -349	+11 / -53	+158 / -401	7 / 6	+10 / -23	+168 / -424	+96	-1225	+110 / -94	+404 / -475	+245 / -281	-1075	-1593	+500 / -379	-1506	+455 / -1593	44.08	Same as U1U2	(Si)	do	do	G. 71.4 N. 59.5
U3U4	-391	+308 / -474	+23 / -66	+331 / -540	7 / 6	+22 / -30	+352 / -570	+654	-1550	+286 / -284	+833 / -898	+557 / -591	+117 / -1000	+939 / -2031	+1487 / -244	-2141	+1352 / -2031	44.08	1-C.P. 24"×1/2" 23" 4-Ls 6"×4"×1/2" 2-Webs 30"×5/16"	(Si)	[ ]	rx = 10.9 ry = 10.9	G. 76. N. 62.9
U4U5	-391	+308 / -474	+23 / -66	+331 / -540	7 / 6	+22 / -30	+352 / -570	+654	-1550	+309 / -325	+942 / -1007	+640 / -674	+117 / -1000	+956 / -2072	+1596 / -353	-2224	+1452 / -2072	44.08	Same as U3U4	(Si)	do	do	G. 76. N. 62.9
U5U6	+794	+398 / -284	+29 / -40	+427 / -324	7 / 6	+28 / -18	+455 / -342	+1479	-655	+359 / -336	+1009 / -1000	+655 / -653	+1249	+2127 / -528	+2488	-1308	+2265 / -1190	44.41	1-C.P. 24"×1/2" 22" 4-Ls 6"×4"×1/2" 4 Webs 30"×9/16"	(Si)	[ ]	rx = 10.6 ry = 11.1	G. 98.5 N. 81.4
U6U7	+811	+407 / -290	+30 / -41	+437 / -331	7 / 6	+28 / -19	+464 / -349	+1511	-668	+366 / -340	+1026 / -1017	+666 / -664	+1275	+2171 / -594	+2537	-1332	+2300 / -1210	45.35	Same as U5U6	(Si)	do	do	G. 98.5 N. 81.4
U7U8	+1842	+456 / -75	+25 / -9	+481 / -84	5 / 9	+22 / -7	+503 / -91	+1856	+639	+179 / -170	+663 / -651	+375 / -368	+2345	+3028	+2519	+1014	+3130	46.65	1-C.P. 24"×1/2" 21" 4-Ls 6"×4"×9/16" 4 Webs 30"×5/16"	(Si)	[ ]		G. 123.2 N. 101.6
U8L9	+1901	+471 / -78	+26 / -9	+497 / -86	5 / 9	+23 / -8	+520 / -94	+1915	+659	+184 / -175	+683 / -670	+387 / -377	+2421	+3125	+2598	+1046	+3240	48.13	Same as U7U8	(Si)	do		G. 123.2 N. 101.6
L0U1	-601	+101 / -280	+11 / -64	+112 / -344	7 / 6	+7 / -19	+119 / -363	-2	-1003	+119 / -117	+225 / -226	+173 / -174	-964	-1449	+223 / -228	-1177	+203 / -1443	64.70	4-Ls 6"×4"×3/8" 2-Webs 30"×1/2" 2-Webs 30"×3/8"	(Si)	[ ]	rx = 9.9 ry = 11.6	G. 66.9 N. 54.5
U1L2	+314	+212 / -110	+53 / -	+264 / -121	6 / 6	+16 / -7	+280 / -128	-127	+704	+117 / -121	+226 / -222	+174 / -166	+594	+1009 / -175	+99 / -349	+878	+1009 / -317	51.45	4-Ls 6"×4"×3/8" 2-Webs 30"×1/2"	(Si)	[ ]		G. 44.4 N. 36.1
L2U3	+8	+144 / -164	+22 / -44	+166 / -208	6 / 6	+10 / -13	+176 / -221	+298	-381	+119 / -113	+231 / -223	+167 / -164	+276 / -307	+658 / -730	+529	-545	+658 / -730	60.98	4-Ls 6"×4"×3/8" 2-Webs 24"×1/2"	(Si)	[ ]	rx = 8.3 ry = 11.5	G. 38.4 N. 30.1
U3L4	-326	+111 / -174	+32 / -31	+143 / -204	7 / 6	+10 / -11	+153 / -215	-438	-53	+108 / -115	+221 / -219	+162 / -161	-541	+225 / -911	-657	+215 / -108	+225 / -911	56.96	4-Ls 6"×4"×1/2" 2-Webs 24"×1/2"	(Si)	[ ]	rx = 8.5 ry = 11.4	G. 43. N. 35.6
L4U5	+705	+223 / -61	+49 / -19	+272 / -79	5 / 7	+14 / -6	+286 / -85	+572	+418	+69 / -79	+159 / -150	+104 / -102	+991	+1346	+731	+522	+1346	59.80	4-Ls 6"×4"×1/2" 2-Webs 27"×3/8"	(Si)	[ ]		G. 52.8 N. 42.1
U5L6	-872	+16 / -228	+5 / -58	+21 / -286	8 / 5	+2 / -14	+22 / -300	-519	-785	+66 / -79	+170 / -171	+134 / -137	-1172	-1651	-690	-922	-1562	58.27	4-Ls 6"×4"×3/8" 4 Webs 30"×5/16"	(Si)	[ ]	rx = 9.7 ry = 11.5	G. 66.9
L6U7	+887	+276 / -49	+70 / -5	+346 / -54	6 / 7	+19 / -4	+365 / -57	+312	+1138	+154 / -164	+313 / -313	+226 / -241	+1252	+1771	+625 / -1	+1364	+1771 / -1	70.59	4 Ls 6"×4"×3/8" 4 Webs 30"×1/2"	(Si)	[ ]		G. 74.4 N. 60.7
U7L8	-648	+67 / -238	+7 / -55	+74 / -293	7 / 6	+5 / -16	+79 / -	147	-919	+163 / -156	+289 / -289	+224 / -224	-958	-1423	+142 / -436	-1143	+129 / -1423	71.61	4 Ls 6"×4"×1/2" 4 Webs 30"×1/2"	(Si)	[ ]	rx = 9.9 ry = 11.4	G. 79. N. 64.2
L8U9	+619	+258 / -96	+58 / -10	+315 / -106	6 / 7	+18 / -7	+339 / -113	+69	+991	+186 / -194	+338 / -339	+260 / -257	+952	+1470	+407 / -270	+1251	+1470 / -245	101.53	4 Ls 6"×4"×3/8" 2 Webs 30"×3/8"	(Si)	[ ]		G. 59.4 N. 48.4
L0U0	-85	0 / -55	0 / -50	10 / -104	0 / 30	0 / -32	0 / -136	-190	-90	-56	-70	-61	-221	-412	-200	-151	-412	44.80	2-15" [40#	(Si)	[ ]	rx = 5.4 ry = 11.5	G. 23.4
L1U1	+16	0 / 0	0 / 0	0 / 0	0 / 0	0 / 0	0 / 0	+16	+17	+2	-1	-1	+16	+18	+16	+17	+21	42.19	2-15" [33.9#		[ ]		G. 19.8 N. 15.5
L2U2	-171	0 / -55	0 / -50	0 / -104	0 / 24	0 / -25	0 / -129	-160	-176	0	-1	-1	-300	-428	-161	-177	-428	39.58	2-15" [40#	(Si)	[ ]	rx = 5.4 ry = 11.5	G. 23.4 N. 18.5
L3U3	+30	0 / 0	0 / 0	0 / 0	0 / 0	0 / 0	0 / 0	+30	+32	-2	-5	-3	+30	+30	+30	+32	+40	38.72	2-15" [33.9#		[ ]		G. 19.8 N. 15.5
L4U4	-175	0 / -55	0 / -50	0 / -104	0 / 24	0 / -25	0 / -129	-164	-181	+2	-1	-1	-304	-432	-165	-182	-432	37.86	2-15" [40#	(Si)	[ ]	rx = 5.4 ry = 11.5	G. 23.4
L5U5	+32	0 / 0	0 / 0	0 / 0	0 / 0	0 / 0	0 / 0	+32	+32	-61	-113	-60	+32	+32 / -29	+32 / -81	+32 / -28	+43 / -74	39.35	2-15" [40#		[ ]	rx = 5.4 ry = 11.5	G. 23.4 N. 18.5
L6U6	+63 / -83	+45 / -87	+5 / -56	+50 / -142	7 / 6	+3 / -8	+53 / -150	+138 / -161	-248	+41 / -49	+85 / -97	+56 / -72	+169 / -233	+295 / -457	+223 / -258	-320		44.20	2 Webs 24"×1/2" 4 Ls 6"×4"×1/2"	(Si)	[ ]	rx = 8.6 ry = 11.5	G. 43. N. 36.2
L7U7	+180 / +34	+55 / 0	+50 / 0	+104 / 0	24 / 0	+25	+129 / 0	+66	+184	-35	+2 / -21	+2 / -25	+309	+437	+68	+186		55.85	4 Webs 24"×1/2" 4 Ls 6"×4"×1/2"	(Si)	[ ]	rx = 8.0 ry = 11.3	G. 67. N. 53.8
L8U8	+124 / -22	+39 / -61	+3 / -51	+42 / -112	5 / 9	+2 / -10	+44 / -121	+122 / -169	+22 / -128	+16 / -45	+58 / -82	+29 / -62	+185 / -158	+312 / -430	+180 / -251	+51 / -190		72.00	4 Ls 6"×4"×3/8" 2 Webs 24"×1/2"	(Si)	[ ]	rx = 8.4 ry = 11.4	G. 38.4 N. 30.1
L9U9	+1305 / -1451	+1 / -301	+1 / -49	+2 / -350	18 / 4	0 / -12	+2 / -363	-288	-1180	+374 / -366	+286 / -420	+219 / -285	-1814	-2542	-708	-1465	-2542	93.00	4 Ls 6"×6"×3/4" 8 Webs 36"×5/16" 3/4" 6	(Si)	[ ]	rx = 11.4 ry = 10.7	G. 195.8

Members in Span 1

Bar	Dead Stress	Live Stress Unif.	Live Stress Conc.	Live Stress Total	% I	Imp.	Live & Imp.	Erection Dead + Trav. A	Erection Dead + Trav. B	Wind Comp. Struct.	Wind Erection A	Wind Erection B	D+L+I	D+2(L+I)+W	Total Erect. A	Total Erect. B	Design Stress Basis of 32,400	Length	Make-up of Sections		Assembly	Radius of Gyration	Area
L9L10	-2800	+71 / -670	+8 / -33	+78 / -702	10 / 4	+8 / -31	+86 / -734	-2475		+180 / -457	+19 / -177		-3334	-4755	-2652		-4755	61.17	4 Flg Ls 8"×6"×3/4" 6 Webs 36"×5/8" 1 Diaphragm 19"×3/4"	(Si)	[ ]	rx = 10.4 ry = 10.0	G. 261.8
L10L11	-814	+126 / -341	+25 / -33	+149 / -374	7 / 5	+10 / -20	+159 / -394	-1652		+103 / -119	+123 / -136		-1208	-1721	-1788		-1721	53.94	4 Ls 6"×6"×9/16" 4 Webs 36"×5/8"	(Si)	[ ]	rx = 12.0 ry = 11.0	G. 118.4
L11L12	-744	+114 / -312	+23 / -31	+137 / -343	7 / 5	+10 / -19	+146 / -362	-1512		+152 / -160	+35 / -47		-1106	-1628	-1559		-1628	49.33	4 Ls 6"×6"×1/2" 4 Webs 36"×1/2"	(Si)	[ ]	rx = 12.2 ry = 11.2	G. 95.0
L12L13	+1002	+428 / -270	+52 / -26	+480 / -296	7 / 5	+35 / -15	+514 / -311	-646		+367 / -410	+30 / -45		+1516	+2397		-691	+2397 / -628	46.68	4 Ls 6"×4"×1/2" 4 Webs 36"×1/2"	(Si)	[ ]	rx = 12.1 ry = 11.6	G. 95.4 N. 77.4
L13L14	+976	+417 / -263	+50 / -25	+467 / -288	7 / 5	+34 / -15	+501 / -303	-630		+388 / -424	+4 / -43		+1477	+2366		-673	+2366 / -612	45.45	Same as L12L13	(Si)	do	do	G. 95.4 N. 77.4
L14L15	+1970	+632 / -277	+70 / -19	+702 / -296	7 / 5	+51 / -16	+752 / -312	-54		+544 / -573	0 / -32		+2722	+4018		-86	+4018 / -78	44.55	4 Ls 8"×6"×5/8" 4 Webs 36"×1/2" 2 Webs 36"×5/8"	(Si)	[ ]		G. 154.9 N. 127.9
L15L16	+1946	+625 / -274	+69 / -19	+694 / -293	7 / 5	+50 / -16	+744 / -309	-53		+549 / -572	-2		+2690	+3983		-55	+3983 / -50	44.05	Same as L14L15	(Si)	do		G. 154.9 N. 127.9
U9U10	+1646	+437 / -63	+29 / -10	+466 / -73	7 / 9	+31 / -7	+497 / -80	+1851		+63 / -127	+401 / -482		+2133	+2683	+2252		+2844	49.42	1 C.P. 4 Webs 4 Webs 30"×	(Si)	[ ]		G. 107.4 N. 88.9
U10U11	+1621	+430 / -62	+28 / -10	+458 / -72	5 / 9	+21 / -7	+479 / -79	+1826		+132 / -187	+315 / -381		+2100	+2711	+2141		+2800	48.75	Same as U9U10	(Si)	do		G. 107.4 N. 88.9
U11U12	-199	+260 / -253	+28 / -39	+288 / -292	5 / 8	+14 / -24	+302 / -318	+1054		+53 / -44	+166 / -218		+245 / -599	+751 / -1112	+1220		+1110 / -1112	46.99	1 C.P. 24"×1/2" 23" 4 Ls 6"×4"×3/8" 2 Webs 30"×3/8"	(Si)	[ ]	rx = 10.8 ry = 11.0	G. 71.4 N. 59.4
U12U13	-194	+254 / -249	+27 / -37	+281 / -286	5 / 8	+14 / -24	+295 / -310	+1030		+52 / -60	+108 / -148		+239 / -584	+733 / -1101	+1138		+1032 / -1101	45.88	Same as U11U12	(Si)	do	do	G. 71.4 N. 59.4
U13U14	-1577	+271 / -549	+22 / -62	+293 / -611	5 / 7	+15 / -44	+308 / -655	+304		+142 / -151	+54 / -82		-2292	-3039		+358	+926 / -3039	44.91	1 C.P. 24"×1/2" 4 Ls 8"×6"×1/2" 6 Webs 30"×3/8"	(Si)	[ ]	rx = 10.7 ry = 10.7	G. 117.8 N. 98.7
U14U15	-1556	+268 / -545	+22 / -63	+290 / -608	5 / 7	+15 / -44	+305 / -650	+300		+123 / -127	+21 / -40		-2206	-2983		+321	+291 / -2983	44.34	Same as U13U14	(Si)	do	do	G. 117.8 N. 98.7
U15U16	-2081	+277 / -656	+16 / -72	+293 / -728	5 / 7	+15 / -53	+308 / -781	0		+194 / -182	0 / -8		-2862	-3825	-8		-3825	44.05	1 C.P. 24"×1/2" 4 Ls 8"×6"×1/2" 8 Webs 30"×7/16"	(Si)	[ ]	ry = 10.8	G. 144.
U9L10	+826	+209 / -62	+38 / -8	+247 / -70	7 / 5	+18 / -4	+265 / -74	+201		+133 / -129	+66 / -71		+1091	+1489	+267		+1489	66.98	4 Ls 6"×4"×5/8" 2 Webs 30"×7/16"	(Si)	[ ]		G. 58. N. 47.1
L10U11	-1890	+72 / -441	+7 / -73	+80 / -514	5 / 7	+4 / -37	+84 / -551	-708		+238 / -235	+104 / -103		-2441	-3227	-811		-3256	103.79	4 Ls 8"×6"×1" 8 Webs 36"×5/8"	(Si)	[ ]	rx = 11.3 ry = 10.5	G. 232.
U11L12	+1154	+271 / -38	+46 / -6	+317 / -44	5 / 8	+16 / -3	+333 / -47	+491		+158 / -160	+31 / -35		+1487	+1978	+522		+1984	59.80	4 Ls 6"×4"×5/8" 2 Webs 30"×9/16" 2 Webs 30"×5/16"	(Si)	[ ]		G. 82.8 N. 67.3
L12U13	-1426	+74 / -364	+9 / -61	+83 / -425	7 / 5	+6 / -22	+89 / -447	-708		+185 / -182	+19 / -18		-1873	-2502	-726		-2502	82.68	4 Ls 8"×6"×5/8" 4 Webs 30"×1/2" 2 Webs 30"×1/2"	(Si)	[ ]	rx = 9.9 ry = 10.8	G. 142.2
U13L14	+839	+242 / -71	+43 / -10	+285 / -81	5 / 7	+15 / -6	+301 / -87	+435		+120 / -121	0 / -2		+1140	+1561	+435		+1561	61.52	4 Ls 6"×4"×1/2" 2 Webs 30"×3/8"	(Si)	[ ]		G. 64. N. 51.9
L14U15	-658	+113 / -244	+19 / -46	+132 / -290	7 / 6	+9 / -16	+141 / -306	-403		+94 / -87	+6 / 4		-964	-1357	-407		-1357	72.40	4 Ls 6"×4"×1/2" 2 Webs 30"×3/8"	(Si)	[ ]	rx = 10.5 ry = 11.4	G. 68.4
U15L16	+202	+178 / -138	+34 / -25	+212 / -163	6 / 6	+12 / -10	+224 / -173	+79		+29 / -27	0 / -0		+442 / -48	+782 / -335	+79		+782 / -335	65.48	4 Ls 6"×4"×3/8" 2 Webs 21"×1/2"	(Si)	[ ]	rx = 7.8	G. 35.4 N. 27.
U16L16	-85 / -231	+2 / -67	+0.4 / -49	+2 / -117	10 / 4	+0.2 / -5	+3 / -122	-160		+6 / -40	+21 / -17		-353	-515	-177		-515	73.00	4 Ls 6"×5"×5/8" 2 Webs 21"×3/8"	(Si)	[ ]	rx = 7.7	G. 37.7
U10L10	+54	+75 / -56	+55 / -8	+130 / -64	7 / 6	+9 / -4	+139 / -68	-214		+17 / -14	+18 / -19		+209 / -47	+398 / -161	-233		+398 / -212	62.80	2-18" [45.8#		[ ]	rx = 6.6	G. 26.8 N. 22.
U11L11	-14	+19 / -19	+4 / -3	+23 / -22	8 / 5	+2 / -1	+25 / -23	-132		+3 / -16	+13 / -12		+24 / -45	+63 / -96	-144		+63 / -130	57.00	2-18" [45.8#		[ ]	rx = 6.6	G. 26.8 N. 22.
U12L12	+281	+93 / -24	+56 / -3	+149 / -27	7 / 5	+10 / -1	+159 / -28	-13		+35 / -33	0 / -7		+440	+632	-20		+632 / -18	54.40	2-18" [45.8#	(Si)	[ ]		G. 26.8 N. 22.
U13L13	+78	+43 / -21	+7 / -3	+50 / -24	7 / 5	+3 / -1	+53 / -25	-148		+10 / -10	+3 / -4		+131	+194	-152		+194 / -158	52.00	2-15" [33.9#		[ ]	rx = 5.6	G. 19.8 N. 15.7
U14L14	+424	+126 / -31	+61 / -3	+186 / -34	7 / 5	+13 / -2	+199 / -36	+51		+52 / -64	0 / 0		+623	+874	+51		+874	50.50	2-15" [50.0# 2 Pls 12"×3/8"	(Si)	[ ]		G. 38.5 N. 30.7
U15L15	+91	+45 / -19	+7 / -2	+52 / -21	7 / 5	+4 / -1	+56 / -22	-27		+12 / -13	0 / 0		+147	+215	-27		+215 / -25	50.00	2-15" [33.9#		[ ]		G. 19.8 N. 15.7
[illegible]	+503	+238	+50	+288	5	+16	+303	+57	+775	+133			+806	+1242									
[illegible]	+3572	+779	+50	+829	4	+36	+865	+2183	+2310	+714			+4437	+6016									

Erection 'B' does not control design of these members

Bar	Mom. in Ft. Kips due to Wind: Comp. Struct.	Erection	Resistance to Buckling
U8L8	19	19	325
U7L7	121	121	472
U9L9	146	146	
L8L9	578	632	
L9L10	862	960	
L10L11	815	895	
L11L12	705	775	

Notes:

The stress in the column marked "Design Stress" equals for each member, the largest of the following three values: $\frac{4}{3}$(D+L+I); D+2(L+I)+W; $\frac{4}{3}$ (total erection stress). See specifications.

The stresses in the columns marked "Erection A" are based on the assumption that the trusses extend from panel points 0 to 16, inclusive; the floor steel (no concrete) is in place from panel points 0 to 11, inclusive; the traveler load is at U_9.

The stresses in the columns marked "Erection B" are based on the assumption that the trusses extend from panel points 0 to 14, inclusive; the floor steel is in place from panel points 0 to 11, inclusive; the traveler load is at U_{13}.

Material listed under make-up of sections and marked (Si) to be of silicon steel.

WAR DEPARTMENT
CORPS OF ENGINEERS U. S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.

HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS

SAGAMORE AND BOURNE BRIDGES
STRESS SHEET - SPANS 1, 2 AND 3.

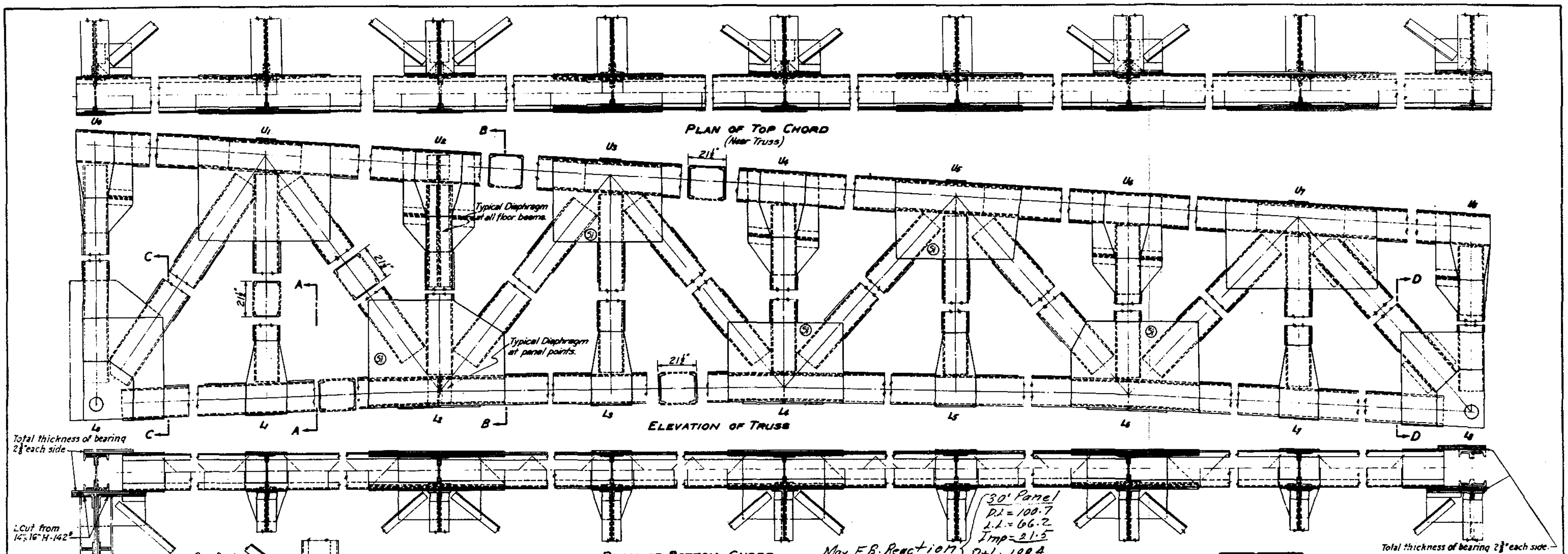
FAY, SPOFFORD & THORNDIKE, ENGINEERS
BY Charles M. Spofford

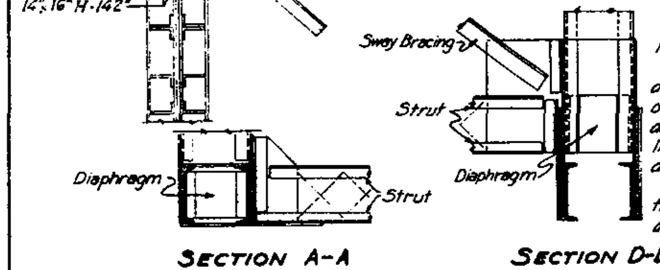
APPROVED R. Park
LT. COL. CORPS OF ENGINEERS
DECEMBER 1933

SUPERSTRUCTURE CONTRACT PLANS SHEET No. 7 OF 17

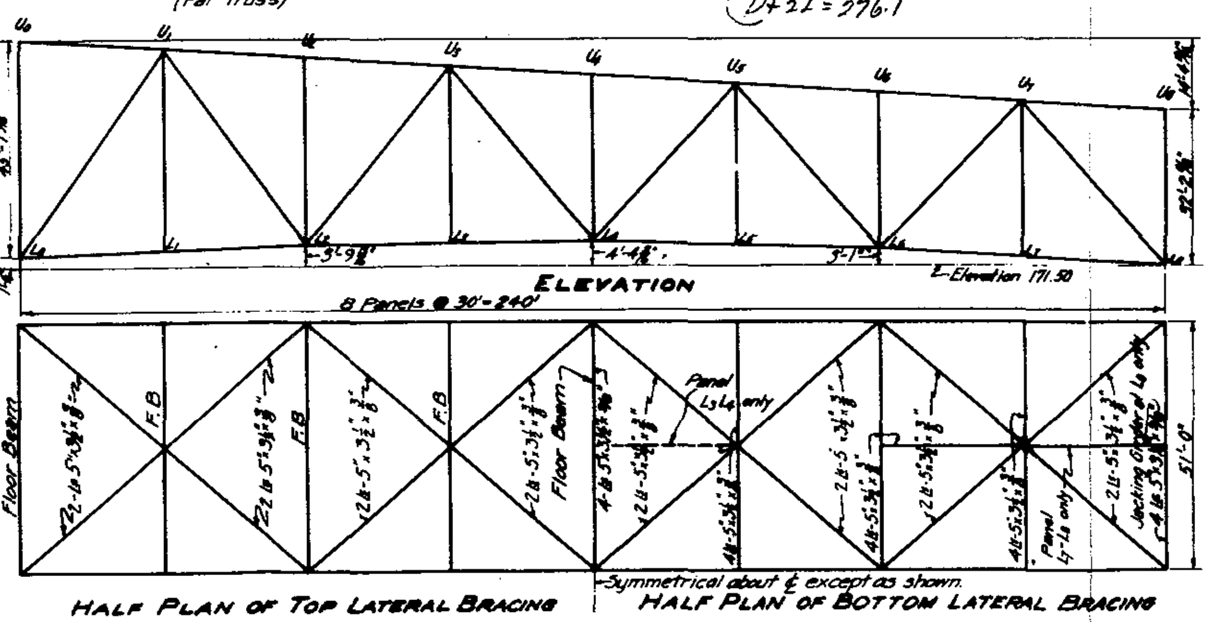
FAY, SPOFFORD & THORNDIKE
BOSTON, MASSACHUSETTS
DESIGNED BY J. B. W.
DRAWN BY W. L. E.
TRACED BY B. T. M.
CHECKED BY I. K. B.

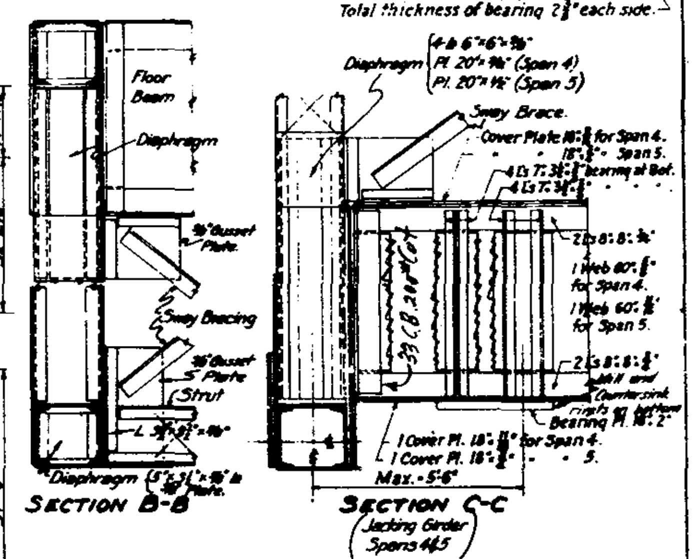
VL-56





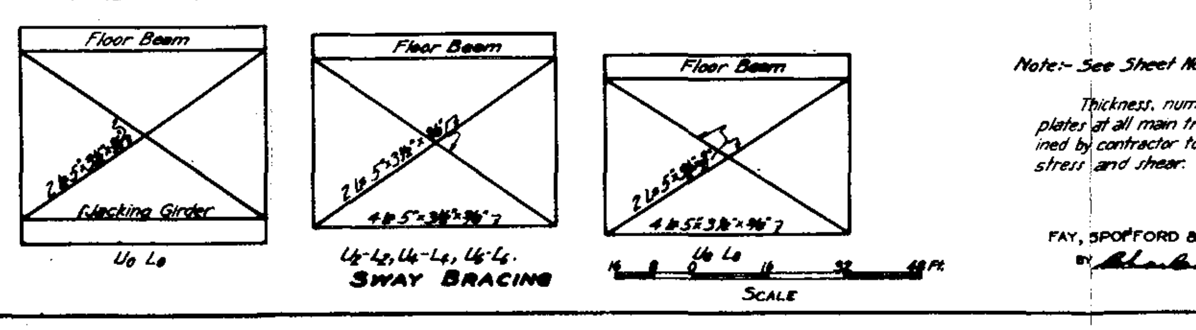
Note:
In the truss diagrams the vertical dimensions shown are computed to the center of the channels of the top chord members and not to the neutral axes or working lines which are to be used in the detailing in accordance with the specifications.
(The vertical dimensions are such as to keep the top of the cover plates of the top chord at a definite distance from the crown of the roadway.)





Member	Stresses in Kips: Dead	Live + Impact	Wind	Design Stress	51.9# Make-up of Section		Area: Gross	Net	Assembly	Radius of Gyration
$U_0 U_1$			-56	-56	2-18" [@ 42.7# Cover Pl. 21"x 9/16"		36.76			
$U_1 U_3$	-600	-239	-93	-1171	2-18" [@ 58# Cover Pl. 21"x 9/16"	(Si)	42.16 (45.77)			r_x = 6.7
$U_3 U_5$	-856	-358	-93	-1665	2-18" [@ 58# Cover Pl. 21"x 9/16" - 2 Pl. [redacted] 17½ x ⅞	(Si)	63.76			r_x = 6.5
$U_5 U_7$	-686	-288	-93	-1355	2-18" [@ 42.7# Cover Pl. 21"x 9/16" 2 Pl. [redacted] 17½ x 7/16	(Si)	50.26			r_x = 6.7
$U_7 U_8$			-56	-56	2-18 [@ 42.7# Cover Pl. 21"x 9/16"		36.76			
$L_0 L_2$	+310	+190		+587	2-18" [@ 42.7#	(Si)	24.96	22.26		
$L_2 L_4$	+758	+317	+16	+1432	2-18" [@ 58# 2-Pl. 18"x ⅝"	(Si)	56.46	45.98		
$L_4 L_6$	+814	+343	+16	+1542	2-18" [@ 58# 2-Pl. 18"x ¾"	(Si)	60.96	49.76		
$L_6 L_8$	+417	+175		+790	2-18" [@ 51.9#	(Si)	30.36	26.76		
$L_0 U_1$	-570	-247		-1090	2-18" [@ 58# 4-Pl. 18"x 9/16"	(Si)	65.48			r_x = 5.9
$U_1 L_2$	+448	+220		+891	2-18" [@ 58#	(Si)	33.96	29.76		
$L_2 U_3$	-289	-171		-631	2-18" [@ 58#	(Si)	33.96			r_x = 6.5
$U_3 L_4$	+157	+137		+431	2-15" [@ 35#	(Si)	20.46	16.08		
$L_4 U_5$	+60	±100 / -86		+325 / -195	2-15" [@ 33.9#		19.80	17.40		r_x = 5.6
$U_5 L_6$	-194	-156		-506	2-18" [@ 42.7#	(Si)	24.96			r_x = 6.6
$L_6 U_7$	+426	+198		+833	2-18" [@ 58#	(Si)	33.96	27.26		
$U_7 L_8$	-620	-259		-1172	2-18" [@ 58# 4-Pl. 18"x 9/16"	(Si)	60.96			r_x = 5.8
$U_0 L_0$, $U_8 L_8$	-63	-112	-60	-347	2-15" [@ 33.9#	(Si)	19.80			r_x = 5.6
$U_2 L_2$	-125	-112	-13	-362	2-15" [@ 33.9# [redacted], $U_4 L_4$, [redacted] $U_6 L_6$, [redacted]	(Si)	19.80			r_x = 5.6
R_0 Gross	528	227	60		2-15" [@ 33.9# $U_1 L_1$, $U_3 L_3$, $U_5 L_5$, $U_7 L_7$					
R_8 Gross	516	227	44							

Notes:
The stress in the column marked "Design Stress" equals for each member, the largest of the following two values: $\frac{4}{3}$(D+L+I) ; D+2(L+I)+W. See Specifications.
Material listed under "Make-up of Section" and marked (Si) to be of silicon steel. Main Truss gusset plates to be of carbon steel unless otherwise designated on the plans. All other material including rivets to be of carbon steel.



Note:- See Sheet No. 14 for additional notes.
Thickness, number and size of gusset plates at all main truss joints to be determined by contractor to carry flexure, direct stress and shear.

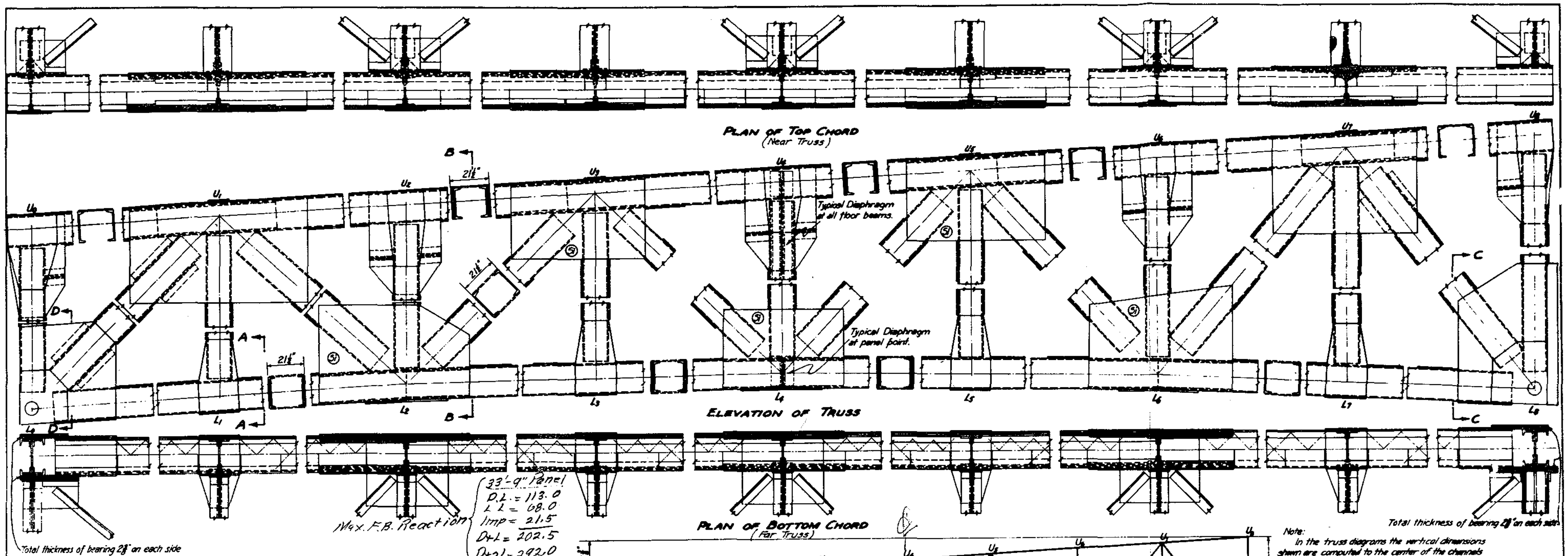
FAY, SPOFFORD & THORNDIKE, ENGINEERS
BY [signature]

WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
OFFICE OF DISTRICT ENGINEER BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
BOURNE BRIDGE
TRUSSES AND BRACING-SPAN 4
SCALE EXCEPT AS SHOWN
APPROVED [signature]
LT. COL. CORPS OF ENGINEERS
DECEMBER 1933
SUPERSTRUCTURE CONTRACT PLANS SHEET No. 15 OF 17

Designed by W.L.C.
Drawn by E.H.
Traced by H.L.K.
Checked by L.K.S.

VL-56
C 21

PLATE XX

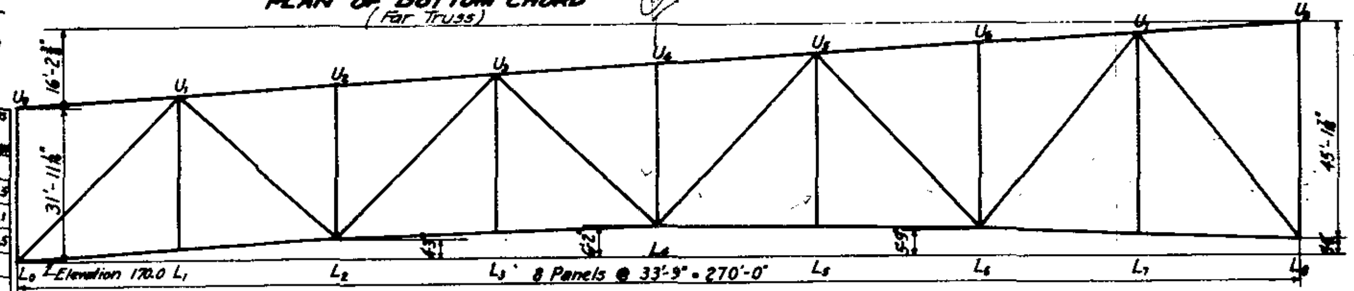


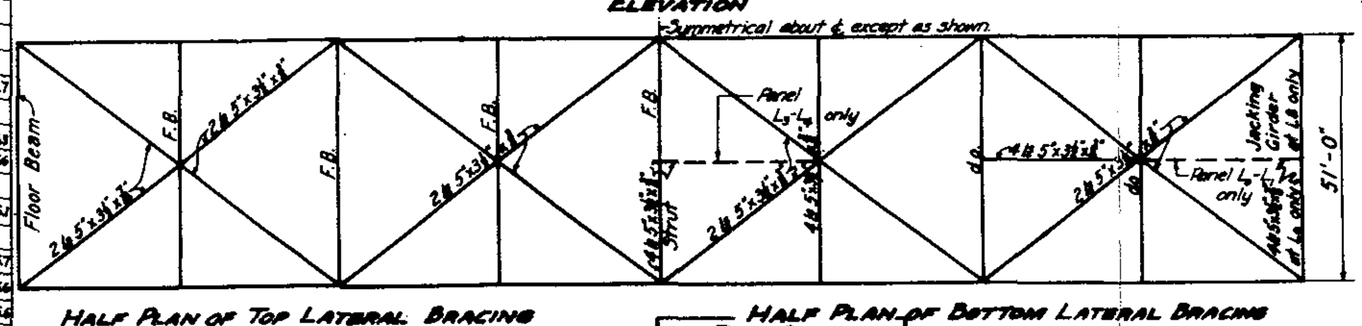
4 Pls 17½ × 11/16"

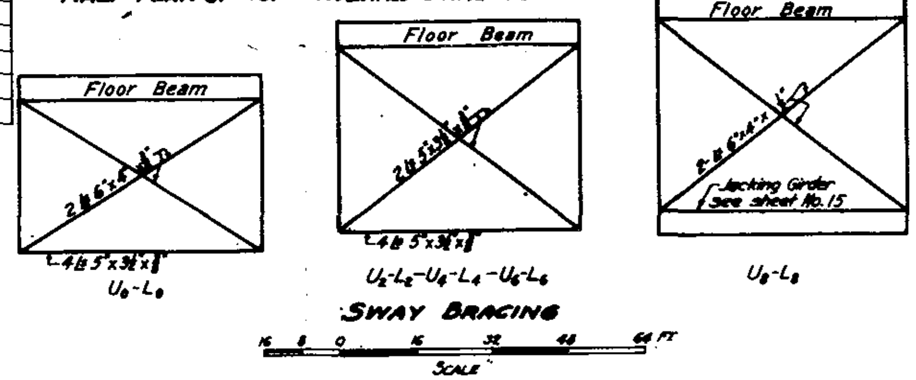
Member	Stress in Kips: Dead	Live + Impact	Wind	Design Stress	Make-up of Section		Area: Gross	Net	Assembly	Radius of Gyration
U_0-U_1			-82	-82	Cover Pl 21"×⅝ 2-⊏-18"@42.7#		36.76			
U_1-U_3	-1000	-363	-117	-1843	Cover Pl 21"×⅝ 4Pl 18"×⅝ 2-⊏-18"@58# – 51.9#	Si	77.3			r_x=6.5
U_3-U_5	-1245	-454	-117	-2270	Cover Pl 21"×⅝ 2Pl 18"×⅝ 2Pl 18"×⅝ 2-⊏-18"@58#	Si	93.01			r_x=6.1
U_5-U_7	-828	-302	-117	-1549	Cover Pl 21"×⅝ 2Pl 18"×⅝ 2-⊏-18"@58# – 51.9#	Si	61.51			r_x=6.5
U_7-U_8			-82	-82	Cover Pl 21"×⅝ 2-⊏-18"@42.7# 17½×9/16		36.76			
L_0-L_2	+579	+212		+1054	45.8 2-⊏-18"@51.9# 2Pl 15"×⅝ – 18×3/8	Si	41.61	35.21		
L_2-L_4	+1200	+439	+20	+2186	2-⊏-18"@58# 4Pl 18"×⅝	Si	83.46	68.5		
L_4-L_6	+1090	+397	+20	+1982	2-⊏-18"@58# 4Pl 18"×⅝	Si	74.46	60.98		
L_6-L_8	+440	+161		+802	2-⊏-18"@51.9#	Si	30.36	24.26		
L_0-U_1	-825	-308		-1511	2-⊏-18"@58# 4Pl 18"×⅝	Si	83.46			r_x=5.7
U_1-L_2	+564	+245		+1078	2-⊏-18"@51.9# 2Pl 15"×⅝	Si	41.61	35.21		
L_2-U_3	-296	-163		-622	2-⊏-18"@58#	Si	33.96			r_x=6.3
* U_3-L_4	+55	+114 / -97		+360 / -232	2-⊏-15"@33.9# 45# changed by A.B. Co. for Erection stresses		19.8	15.7		r_x=5.6
* L_4-U_5	+218	+159		+536	2-⊏-18"@45.8# 58#		26.76	22.26		
U_5-L_6	-398	-198		-795	2-⊏-18"@51.9# 2Pl 18"×⅝	Si	43.86			r_x=6.1
L_6-U_7	+600	+254		+1138	2-⊏-18"@58# 2Pl 15"×⅝	Si	45.21	38.41		
U_7-L_8	-717	-267		-1312	2-⊏-18"@58# 2Pl 18"×⅝ 2Pl 18"×⅝	Si	81.21			r_x=5.7
L_8-U_8	-78	-94	-67	-333	2-⊏-15"@33.9# L_0-U_0	Si	19.8			r_x=5.6
L_2-U_2	-156	-94	-14	-358	2-⊏-15"@33.9# L_4-U_4 L_6-U_6	Si	19.8			r_x=5.6
					2-⊏-15"@33.9# L_1-U_1 L_3-U_3 L_5-U_5 L_7-U_7					
R_0 Gross	536	247	48							
R_8 Gross	538	247	67							

* Chan

Notes:
The stress in the column marked "Design Stress" equals for each member, the largest of the following two values: $\frac{4}{3}(D+L+I)$; $D+2(L+I)+W$. See specifications.
Material listed under "Make-up of Section" and marked (Si) to be of silicon steel. Main Truss gusset plates to be of carbon steel unless otherwise designated on the plans. All other material including rivets to be of carbon steel.







Note:
Thickness, number and size of gusset plates at all main truss joints to be determined by contractor to carry flexure, direct stress and shear.
Latticing.- For members carrying compression only or compression and flexure, latticing is to be proportioned in accordance with the American Association of State Highway Officials Specifications.
For members, the design of which is controlled by tension but which also carry compression, latticing is to be proportioned to carry shear equal to 2½ per cent of the total compression divided equally between the two flanges; double latticing to be used with bars not less than 2½ inches × ⅝ inches.
For members carrying tension only, double latticing to be used with 2½ inches × ⅝ inch bars.

FAY, SPOFFORD & THORNDIKE, ENGINEERS
BY Charles M. Spofford

Note:
In the truss diagrams the vertical dimensions shown are computed to the center of the channels of the top chord members and not to the neutral axes or working lines which are to be used in the detailing in accordance with the specifications.
(The vertical dimensions are such as to keep the top of the cover plates of the top chord at a definite distance from the crown of the roadway.)

Notes:-
For General Plan and Elevations - See sheets Nos. 2 and 3.
For Deck Sections and Deck Details - See Sheets Nos. 4 and 5.
For Truss Bearings - See Sheet No. 17.
For Sections A-A, B-B, C-C, D-D. See Sheet No. 15.
Where consistent with shipping requirements, two or more panel lengths of chord members shall be fabricated as one member.
Joints in compression members shall be faced to bear.
Open holes for field rivets to be 15/16" ϕ unless otherwise noted. 1 1/16 ϕ for all Truss Joints
Compression edges of main gusset plates to be stayed against buckling by angles or plates where unsupported lengths exceed 12 times the plate thickness wherever maximum allowable stress occurs at edge. For stresses less than maximum, unstayed lengths may be increased proportionally.
All rivets ⅞ ϕ.
All elevations are referred to a datum 100 feet below Mean Sea Level.

WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.

HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS

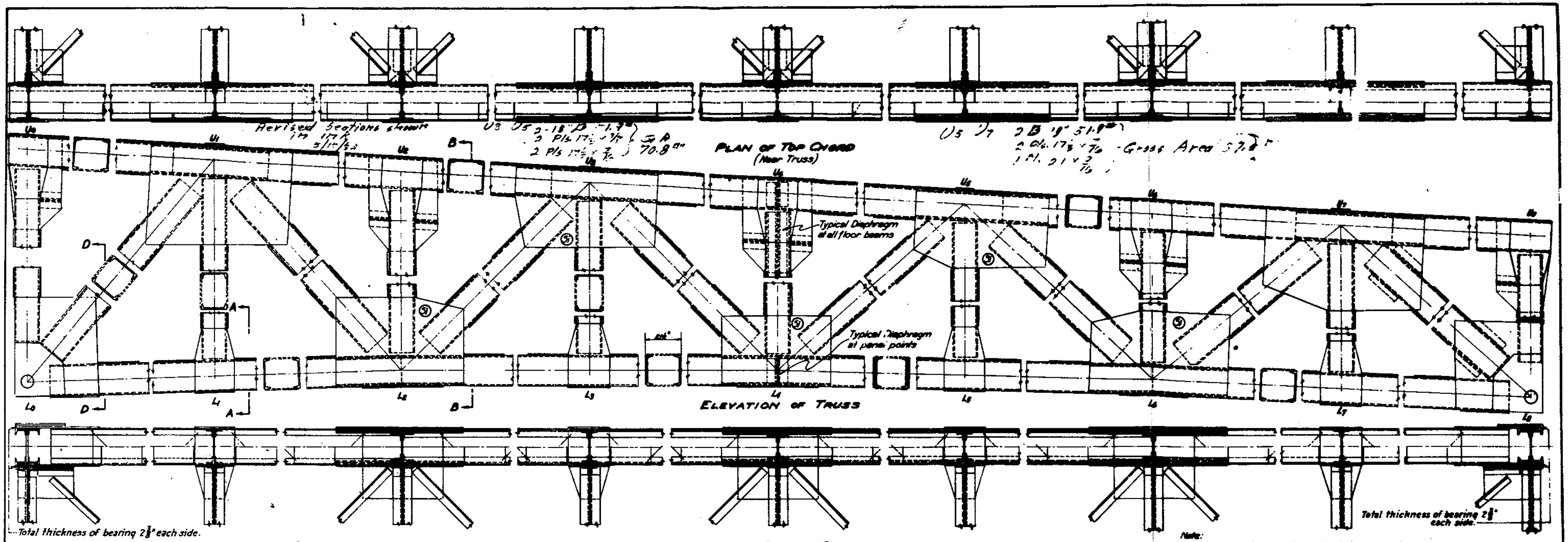
BOURNE BRIDGE
TRUSSES AND BRACING-SPAN 5

SCALE EXCEPT AS SHOWN

APPROVED
LT. COL. CORPS OF ENGINEERS
DECEMBER 1933
SUPERSTRUCTURE CONTRACT PLANS SHEET No. 14 OF (17)

VL-56

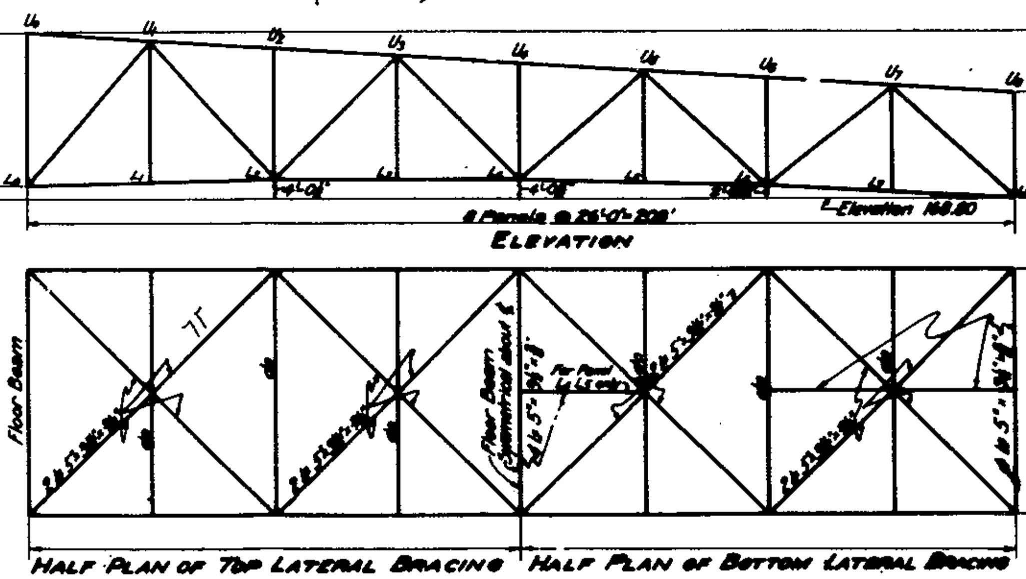
PLATE XXI



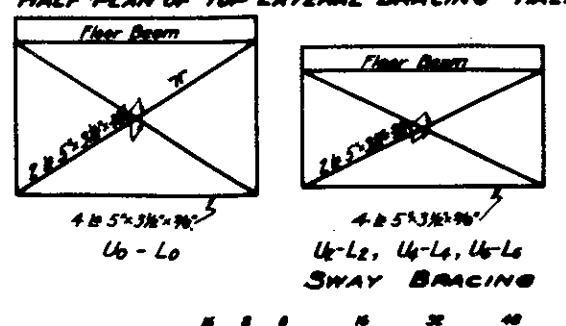
Note:
In the truss diagrams the vertical dimensions shown are computed to the center of the channels of the top chord members and not to the neutral axes or working lines which are to be used in the detailing in accordance with the specifications.
(The vertical dimensions are such as to keep the top of the cover plates of the top chord at a definite distance from the crown of the roadway.)

MEMBER	STRESSES IN KIPS: DEAD	LIVE + IMPACT	WIND	DESIGN STRESS	MAKE-UP OF SECTION			AREA: GROSS	NET	ASSEMBLY	RADIUS OF GYRATION
U_0-U_1			-48	-48	2-18" [@ 42.7#	Cover Pl 21 × 9/16"		36.8			r_x=6.6
U_1-U_3	-651	-269	-63	-1252	2-18" [@ 58#	Cover Pl 21 × 9/16"	17½ × 7/16	(Si) 45.8			
U_3-U_5	-981	-401	-63	-1846	2-18" [@ (58) 51.9	Cover Pl 21 × 9/16"	2 Pl (18 × 9/16) 2 Pls 17½ × 7/16	(Si) 70.5			r_x=6.3
U_5-U_7	-791	-326	-63	-1506	2-18" [@ (58)	Cover Pl 21 × 9/16"	2 Pl (15 × ⅜)	(Si) 57.0			r_x=6.4
U_7-U_8			-48	-48	2-18" [@ 42.7#	Cover Pl 21 × 9/16"	17½ × 7/16	36.8			
L_0-L_2	+348	+144		+657	2-18" [@ 42.7#			(Si) 25.0	20.5		
L_2-L_4	+859	+355	+13	+1620	2-18" [@ 58#	2 Pl 18 × 11/16"		(Si) 58.7	50.4		
L_4-L_6	+951	+393	+13	+1793	2-18" [@ 58#	4 2 Pl 18 × ¾ 7/16"	[redacted]	(Si) 64.3	55.0		
L_6-L_8	+462	+192		+873	2-18" [@ 58#			(Si) 34.0	27.0		
L_0-U_1	-536	-242		-1038	2-18" [@ 58#	2 Pl 15 × 9/16"		(Si) 50.8			r_x=5.7"
U_1-L_2	+455	+220		+901	2-18" [@ 42.7#	2 Pl 15 × ⅜"		(Si) 36.2	31.1		
L_2-U_3	-294	-174		-642	2-18" [@ 45.8#			(Si) 26.8			r_x=6.5
U_3-L_4	+175	+139		+453	2-15" [@ 33.9#			(Si) 19.8	16.6		
L_4-U_5	+43	± 89		+ 312 - 220	2-15" [@ 33.9#			19.8	16.6		
U_5-L_6	-219	-154		-527	2-18" [@ 51.9#			30.4			r_x=6.4
L_6-U_7	+424	+209		+845	2-18" [@ 58#			(Si) 34.0	27.0		
U_7-L_8	-626	-282		-1210	2-18" [@ 58#	2 Pl 18 × ½"		(Si) 52.0			r_x=5.9"
L_0-U_0	-49	-84	-37	-254	2-15" [@ 33.9#	L_8-U_8		19.8			r_x=5.6"
L_2-U_2	-101	-84	-9	-278	2-15" [@ 33.9#	L_1-U_1, L_3-U_3, L_4-U_4, L_5-U_5, L_6-U_6, L_7-U_7		19.8			
R_0 Gross	448	205	37								
R_8 Gross	448	205	27								

Notes:
The stress in the column marked "Design Stress" equals for each member, the largest of the following two values: $\frac{4}{3}(D+L+I)$; $D+2(L+I)+W$; See Specifications.
Material listed under "Make-up of Section" and marked (Si) to be of silicon steel. Main Truss gusset plates to be of carbon steel unless otherwise designated on the plans. All other material including rivets to be of carbon steel.



Note:-
See Sheet No. 14 for additional notes.
For Sections A-A, B-B etc see Sheet No. 15
Thickness, number and size of gusset plates at all main truss joints to be determined by contractor to carry flexure, direct stress and shear.

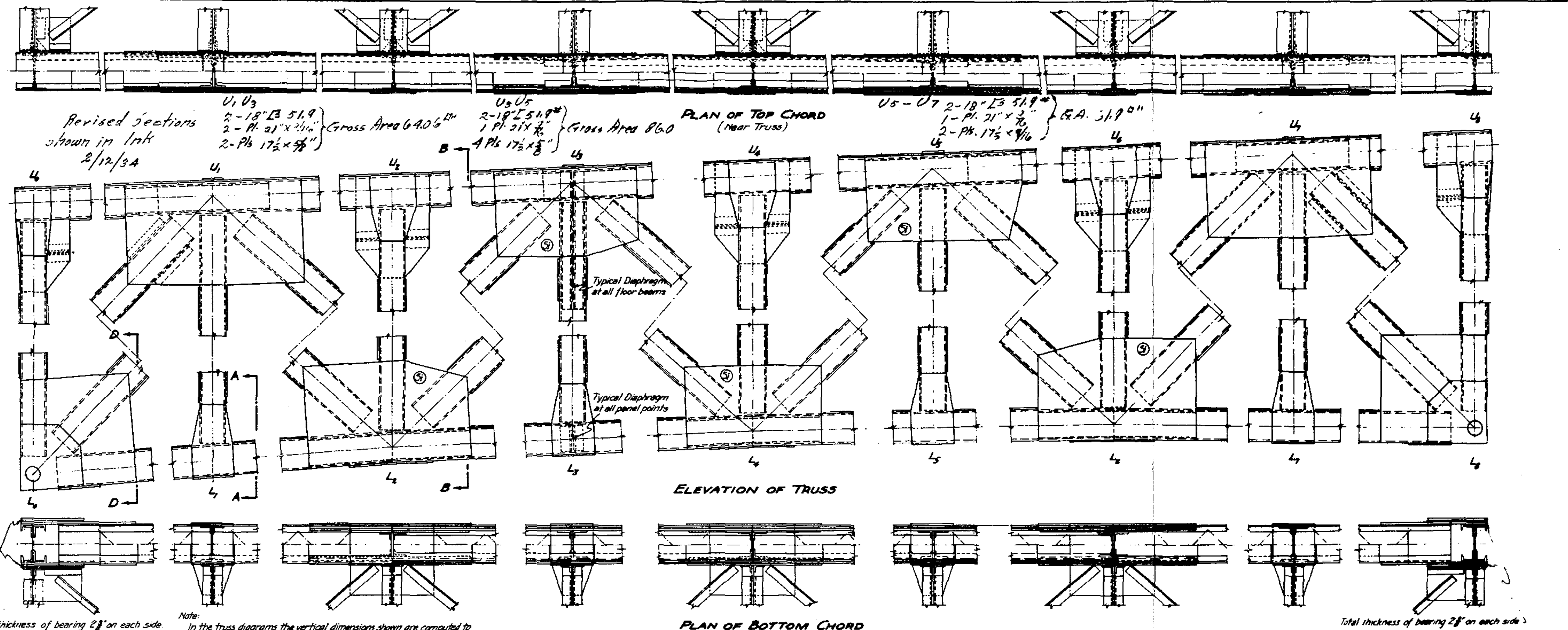


Max. F.B. Reaction
26' Panel
D.L. = 88.6
L.L. = 65.5
Imp = 21.7
D+L = 175.8
D+2L = 263.

FAY, SPOFFORD & THORNDIKE, ENGINEERS
BY Charles M. Spofford

WAR DEPARTMENT
CORPS OF ENGINEERS U. S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.
HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS
BOURNE BRIDGE
TRUSSES AND BRACING - SPAN 6
SCALE EXCEPT AS SHOWN
2 1 0 2 4 6 8 FT.
APPROVED
LT. COL. CORPS OF ENGINEERS
DECEMBER 19[illegible]
SUPERSTRUCTURE CONTRACT PLANS SHEET No. 16 OF (7)

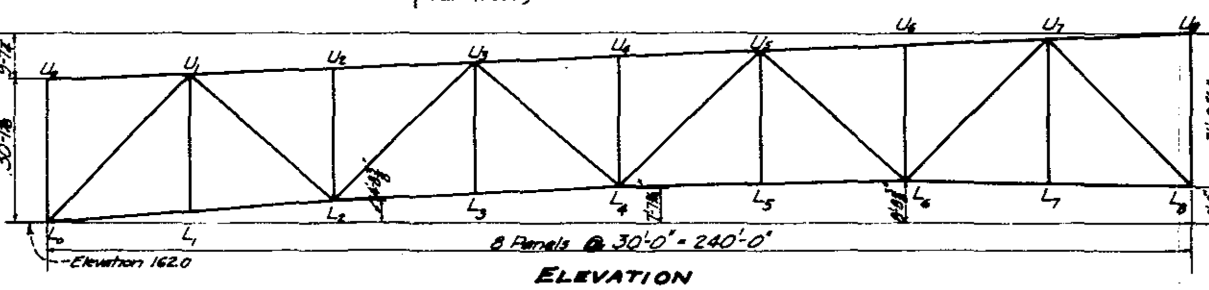
VL-56
C 23

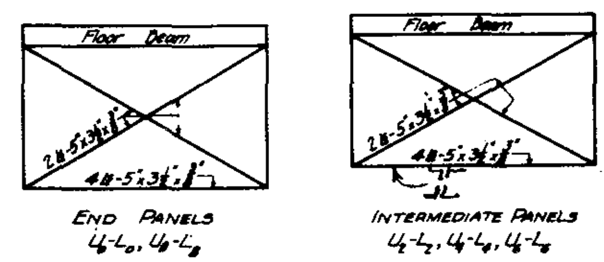


Note:
In the truss diagrams the vertical dimensions shown are computed to the center of the channels of the top chord members and not to the neutral axes or working lines which are to be used in the detailing in accordance with the specifications.
(The vertical dimensions are such as to keep the top of the cover plates of the top chord at a definite distance from the crown of the roadway)

Member	Stresses in Kips: Dead	Live + Impact	Wind	Design Stress	Make-up of Section 51.9#	Area: Gross	Net	Assembly	Radius of Gyration
U_0-U_1			-56	-56	2-18" [@ 42.7# Cover Pl. 21" x 9/16" 17½ 9/16	36.8			
U_1-U_3	-885	-337	-74	-1633	2-18" [@ 58# Cover Pl. 21" x 9/16" 2Pl. 18" x ½" Si	63.8			r_x=6.5"
U_3-U_5	-1205	-461	-74	-2220	2-18" [@ 58# Cover Pl. 21" x 9/16" 4Pl. 18" x 9/16" Si	86.3			r_x=6.2"
U_5-U_7	-863	-329	-74	-1595	2-18" [@ 58# Cover Pl. 21" x 9/16" 2Pl. 18" x 7/16" Si	61.5			r_x=6.5"
U_7-U_8			-56	-56	2-18" [@ 42.7# Cover Pl. 21" x 9/16" 17½ 9/16	36.8			
L_0-L_2	+500	+191		+922	2-18" [@ 58# Si	34.0	29.8		
L_2-L_4	+1120	+428	+16	+2064	2-18" [@ 58# 4Pl. 18" x 5/8" Si	79.0	68.7		
L_4-L_6	+1105	+425	+16	+2040	2-18" [@ 58# 2Pl. 18" x 5/8" 2Pl. 18" x 9/16" Si	76.8	66.7		
L_6-L_8	+478	+182		+880	2-18" [@ 58# Si	34.0	29.8		
L_0-U_1	-722	-299		-1362	2-18" [@ 58# 2Pl. 18" x ½" 2Pl. 18" x 7/16" Si	67.7			r_x=5.8"
U_1-L_2	+518	+235		+1004	2-18" [@ 45.8# 2Pl. 15" x 3/8" Si	38.0	32.0		
L_2-U_3	-328	-186		-700	2-18" [@ 58# Si	34.0			r_x=6.3"
U_3-L_4	+115	+130 / -81		+416 / -123	2-15" [@ 33.9# Si	19.8	17.4		r_x=5.6"
L_4-U_5	+141	+141 / -77		+451 / -24	2-15" [@ 33.9# Si	19.8	17.4		r_x=5.6"
U_5-L_6	-329	-185		-698	2-18" [@ 58# Si	34.0			r_x=6.3"
L_6-U_7	+540	+236		+1035	2-18" [@ 51.9# 2Pl. 15" x 3/8" Si	41.6	35.7		
U_7-L_8	-682	-282		-1286	2-18" [@ 58# 2Pl. 18" x 3/8" 2Pl. 18" x 7/16" Si	63.2			r_x=5.8"
U_0-L_0, U_8-L_8	-69	-112	-43	-336	2-15" [@ 33.9# Si	19.8			r_x=5.6"
U_2-L_2	-137	-112	-10	-371	2-15" [@ 33.9# [redacted] U_4-L_4 [redacted] U_6-L_6, [redacted] Si	19.8			r_x=5.6"
R_0 Gross	470	227	41		2-15" [@ 33.9 U_1L_1, U_3L_3, U_5L_5, U_7L_7				
R_8 Gross	470	227	43						

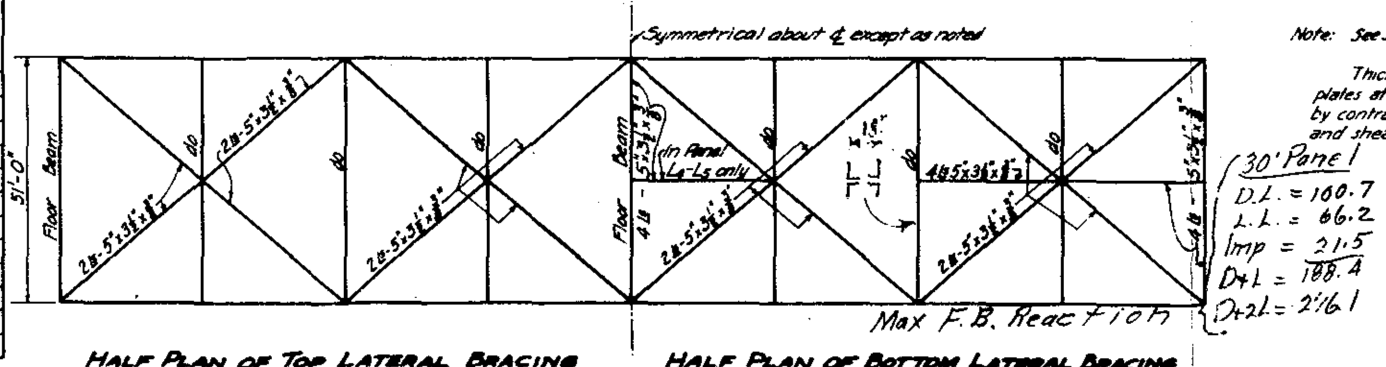
Notes:
The stress in the column marked "Design Stress" equals for each member, the largest of the following two values: $\frac{4}{3}(D+L+I)$; $D+2(L+I)+W$. See specifications.
Material listed under "Make-up of Section" and marked Si to be of silicon steel. Main truss gusset plates to be of carbon steel unless otherwise designated on the plans. All other material including rivets to be of carbon steel.





Note: See Sheet No. 14 for additional notes.

Thickness, number and size of gusset plates at all main truss joints to be determined by contractor to carry flexure, direct stress and shear.



SCALE

WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
OFFICE OF DISTRICT ENGINEER, BOSTON, MASS.

HIGHWAY BRIDGES
OVER CAPE COD CANAL
AT BOURNE, MASSACHUSETTS

BOURNE BRIDGE
TRUSSES AND BRACING - SPAN 7
SCALE EXCEPT AS SHOWN
2 1 0 2 4 6 8 FT

FAY, SPOFFORD & THORNDIKE, ENGINEERS
BY [signature]

APPROVED [signature]
LT. COL. CORPS OF ENGINEERS
DECEMBER 1933
SUPERSTRUCTURE CONTRACT PLANS SHEET No. 12 OF 17

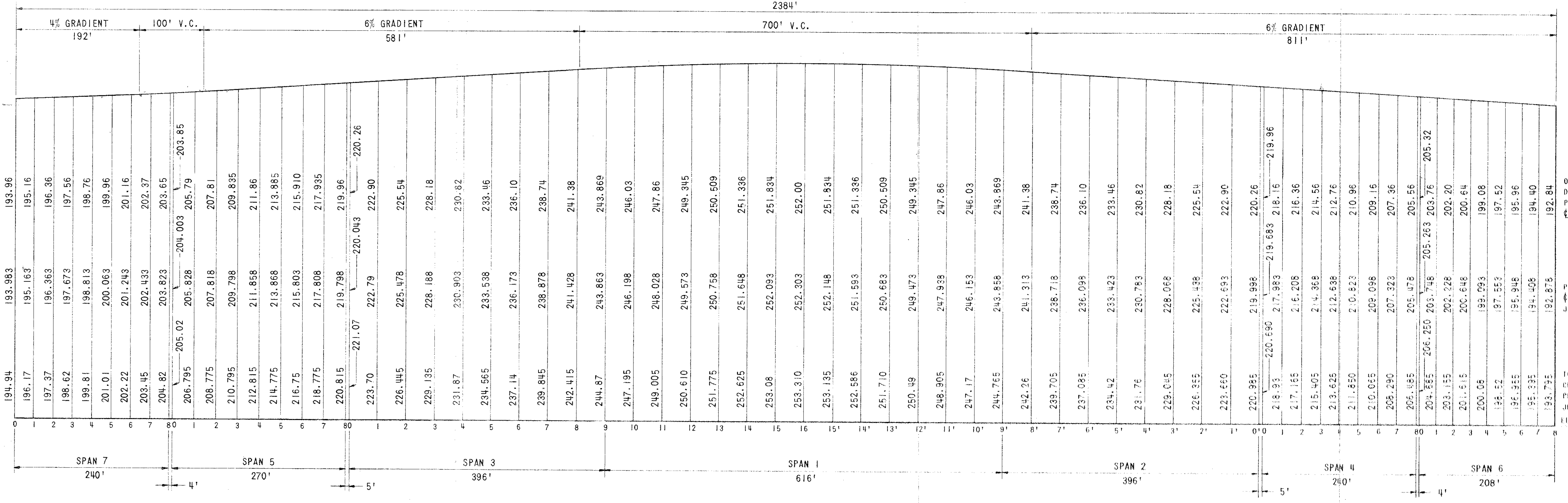
VL-56
C 24

FAY, SPOFFORD & THORNDIKE
DESIGNED BY W.L.C.
DRAWN BY L.W.J.
TRACED BY L.W.J.
CHECKED BY A.T.W.
SCALE: 1"-5' EXCEPT AS SHOWN

PLATE XXII

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.
CAPE COD CANAL
BOURNE, MASS.
BOURNE HIGHWAY BRIDGE
1971 CONDITION REPORT
POSITION OF BEARINGS
JUNE, 1971

VL-56
C-25



Span	Station	Original design profile at ¢ roadway	Profile at ¢ roadway* June 1971	Top of curb profile June 1971
Span 7	0	193.96	193.983	194.94
Span 7	1	195.16	195.163	196.17
Span 7	2	196.36	196.363	197.37
Span 7	3	197.56	197.673	198.62
Span 7	4	198.76	198.813	199.81
Span 7	5	199.96	200.063	201.01
Span 7	6	201.16	201.243	202.22
Span 7	7	202.37	202.433	203.45
Span 7	8	203.65	203.823	204.82
Span 5	0	203.85	204.003	205.02
Span 5	1	205.79	205.828	206.795
Span 5	2	207.81	207.818	208.775
Span 5	3	209.835	209.798	210.795
Span 5	4	211.86	211.858	212.815
Span 5	5	213.885	213.868	214.775
Span 5	6	215.910	215.803	216.75
Span 5	7	217.935	217.808	218.775
Span 5	8	219.96	219.798	220.815
Span 3	0	220.26	220.043	221.07
Span 3	1	222.90	222.79	223.70
Span 3	2	225.54	225.478	226.445
Span 3	3	228.18	228.188	229.135
Span 3	4	230.82	230.903	231.87
Span 3	5	233.46	233.538	234.565
Span 3	6	236.10	236.173	237.14
Span 3	7	238.74	238.878	239.845
Span 3	8	241.38	241.428	242.415
Span 1	9	243.869	243.863	244.87
Span 1	10	246.03	246.198	247.195
Span 1	11	247.86	248.028	249.005
Span 1	12	249.345	249.573	250.610
Span 1	13	250.509	250.758	251.775
Span 1	14	251.336	251.648	252.625
Span 1	15	251.834	252.093	253.08
Span 1	16	252.00	252.303	253.310
Span 1	15'	251.834	252.148	253.135
Span 1	14'	251.336	251.593	252.586
Span 1	13'	250.509	250.683	251.710
Span 1	12'	249.345	249.473	250.49
Span 1	11'	247.86	247.938	248.905
Span 1	10'	246.03	246.153	247.17
Span 1	9'	243.869	243.858	244.765
Span 2	8'	241.38	241.313	242.26
Span 2	7'	238.74	238.718	239.705
Span 2	6'	236.10	236.098	237.085
Span 2	5'	233.46	233.423	234.42
Span 2	4'	230.82	230.783	231.76
Span 2	3'	228.18	228.068	229.045
Span 2	2'	225.54	225.438	226.355
Span 2	1'	222.90	222.693	223.660
Span 2	0'	220.26	219.998	220.985
Span 4	0	219.96	219.683	220.690
Span 4	1	218.16	217.983	218.93
Span 4	2	216.36	216.208	217.165
Span 4	3	214.56	214.368	215.405
Span 4	4	212.76	212.538	213.625
Span 4	5	210.96	210.823	211.850
Span 4	6	209.16	209.098	210.065
Span 4	7	207.36	207.323	208.290
Span 4	8	205.56	205.478	206.985
Span 6	0	205.32	205.263	206.250
Span 6	1	203.76	203.748	204.685
Span 6	2	202.20	202.228	203.155
Span 6	3	200.64	200.648	201.615
Span 6	4	199.08	199.093	200.08
Span 6	5	197.52	197.553	198.52
Span 6	6	195.96	195.948	196.955
Span 6	7	194.40	194.408	195.395
Span 6	8	192.84	192.878	193.795

*DERIVED FROM CURB PROFILES AND CURB HEIGHT MEASUREMENTS.

PROFILE OF BRIDGE

SCALE: HORIZ. 1" = 100'
VERT. 1" = 100'

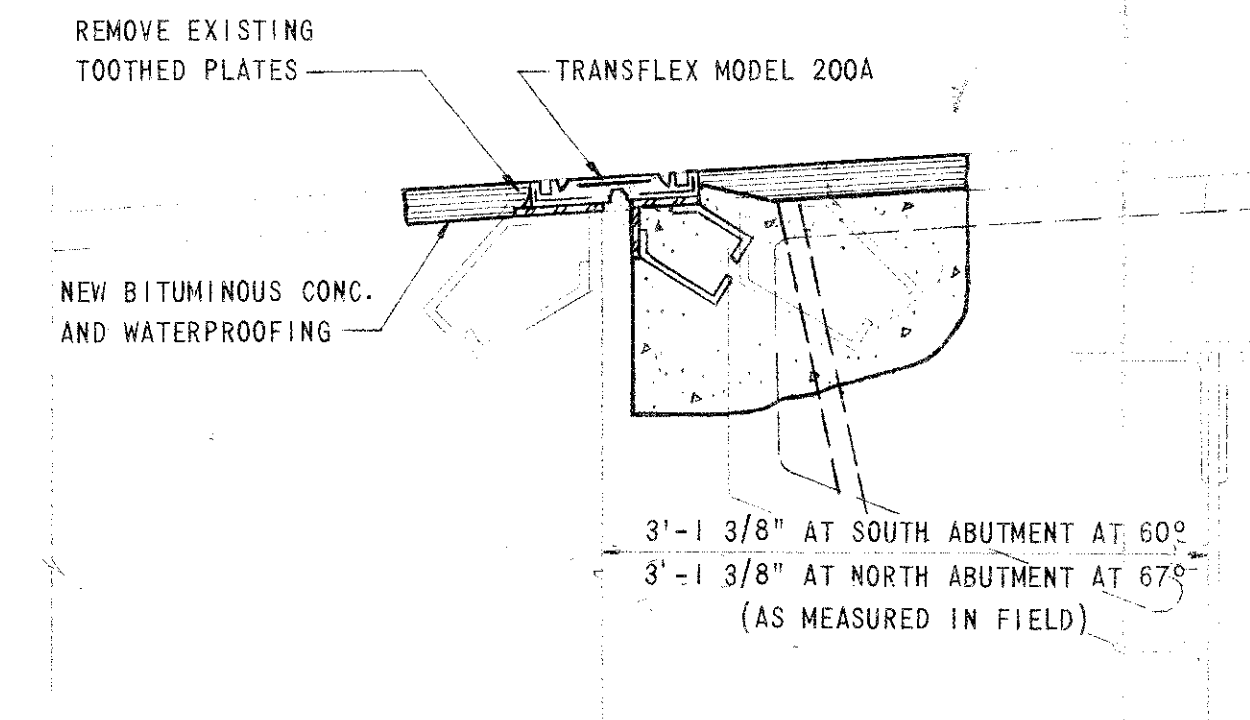
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

CAPE COD CANAL
BOURNE, MASS.

BOURNE HIGHWAY BRIDGE
1971 CONDITION REPORT

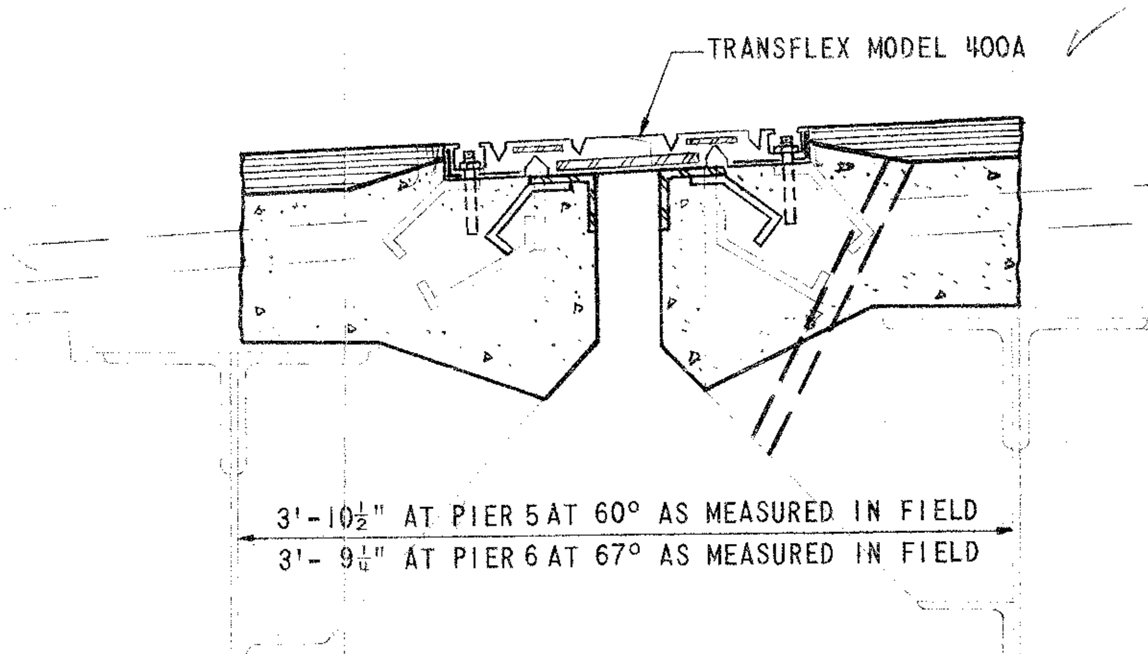
PROFILE SURVEY

FAY, SPOFFORD & THORNDIKE, INC., BOSTON, MASS.

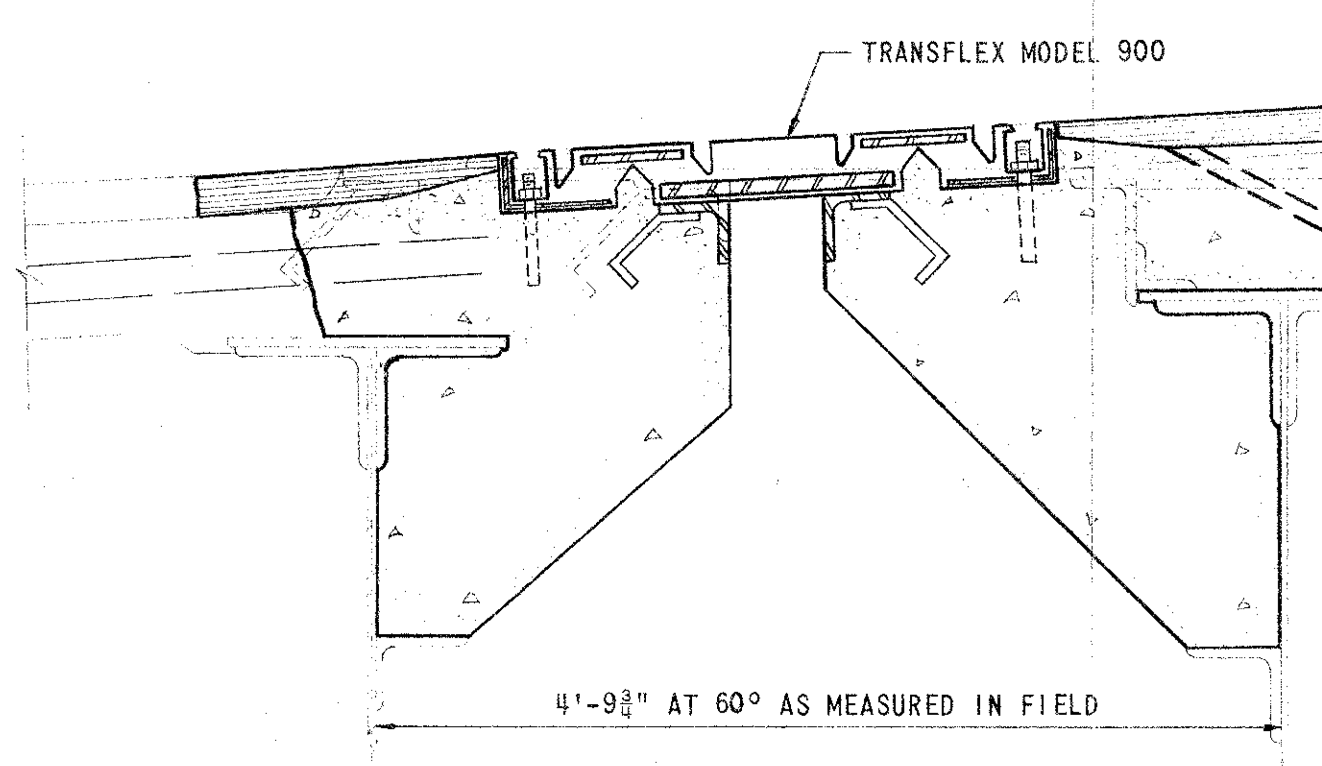


NOTE:
OVERLAY OF NEW EXPANSION JOINT ON EXISTING TOOTHED PLATES IS POSSIBLE IF CHANGE IN PROFILE IS TOLERABLE. THIS APPLIES TO BOTH ABUTMENTS AND JOINTS AT PANEL POINTS 10 AND 10' FOR THIS MODEL OF EXPANSION JOINT ONLY.

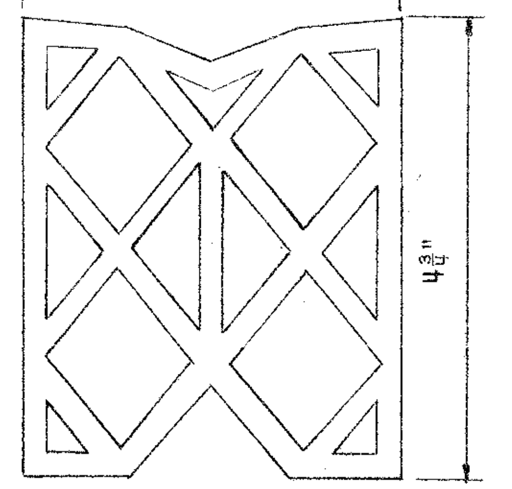
SECTION A-A - SOUTH ABUTMENT
(NORTH ABUTMENT SIMILAR)
TRANSFLEX EXPANSION JOINT MODEL 200A
SCALE: 1" = 1'-0"



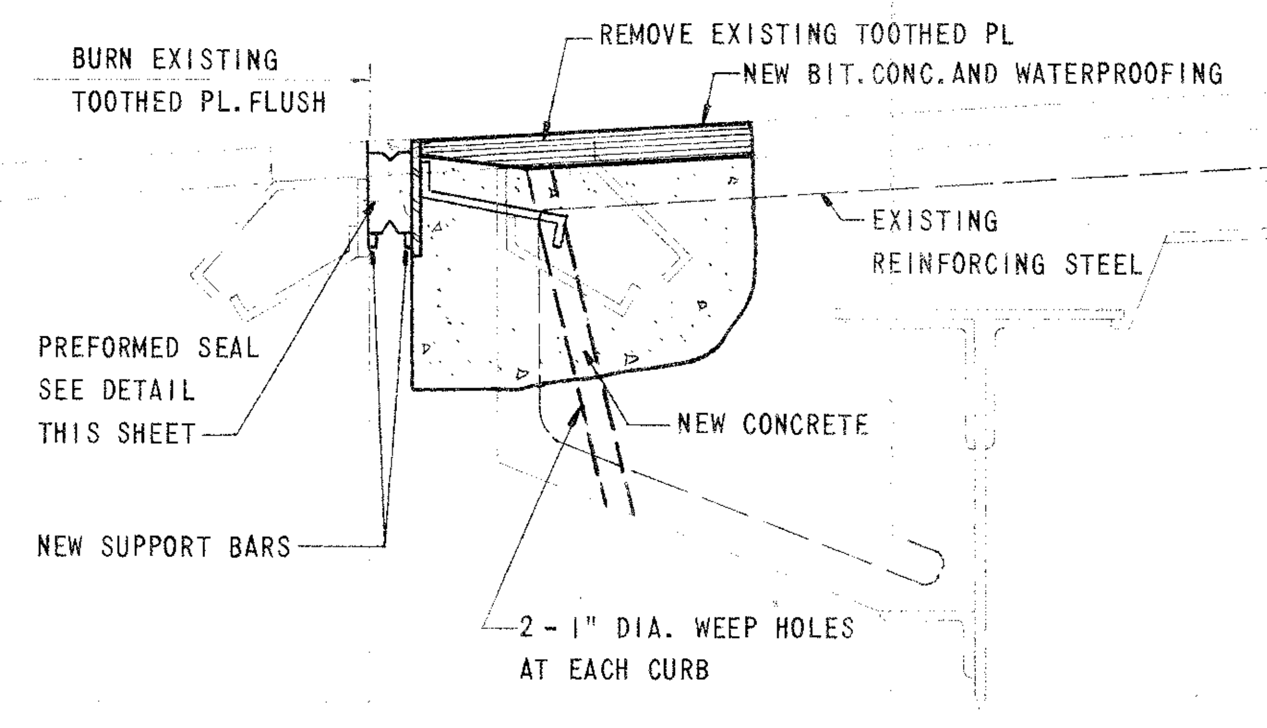
SECTION B-B - PIERS 5 AND 6
TRANSFLEX EXPANSION JOINT MODEL 400A
SCALE: 1" = 1'-0"



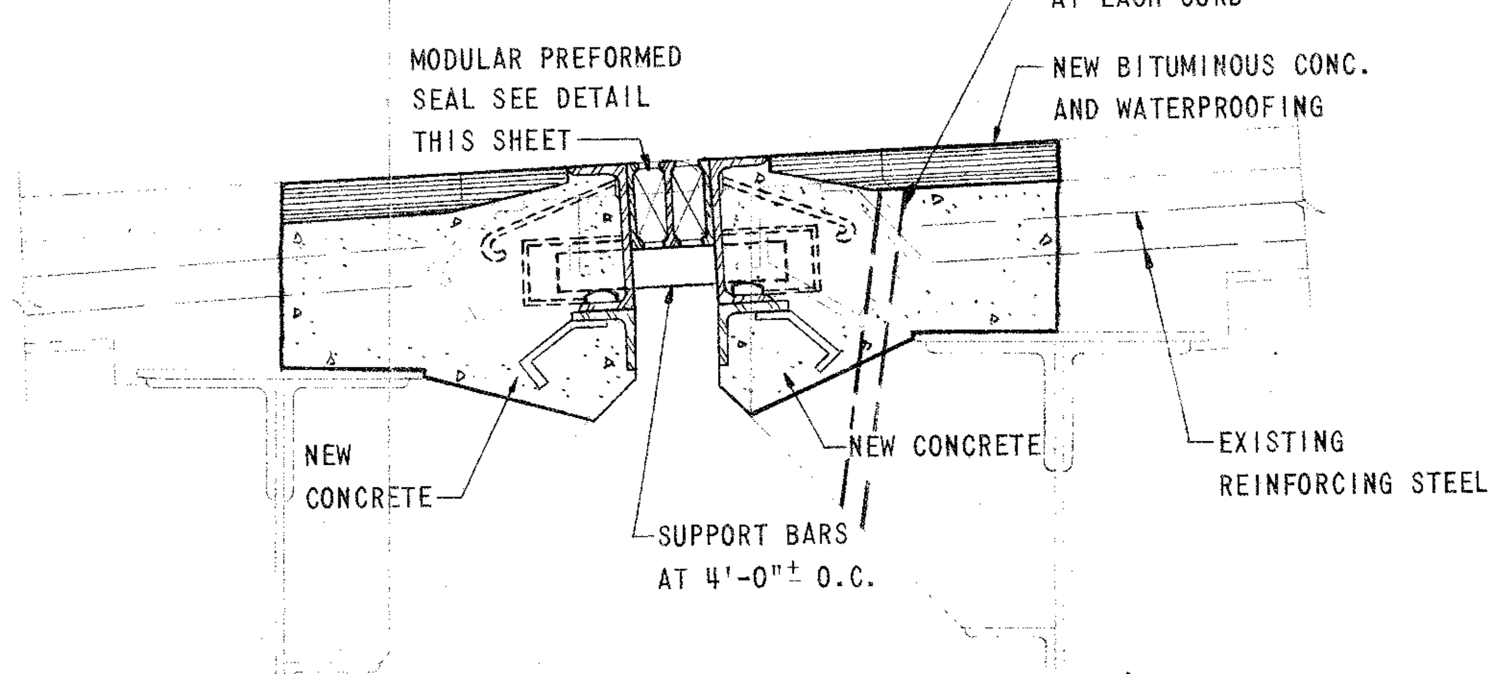
SECTION C-C - PIER 3
TRANSFLEX EXPANSION JOINT MODEL 900
SCALE: 1" = 1'-0"



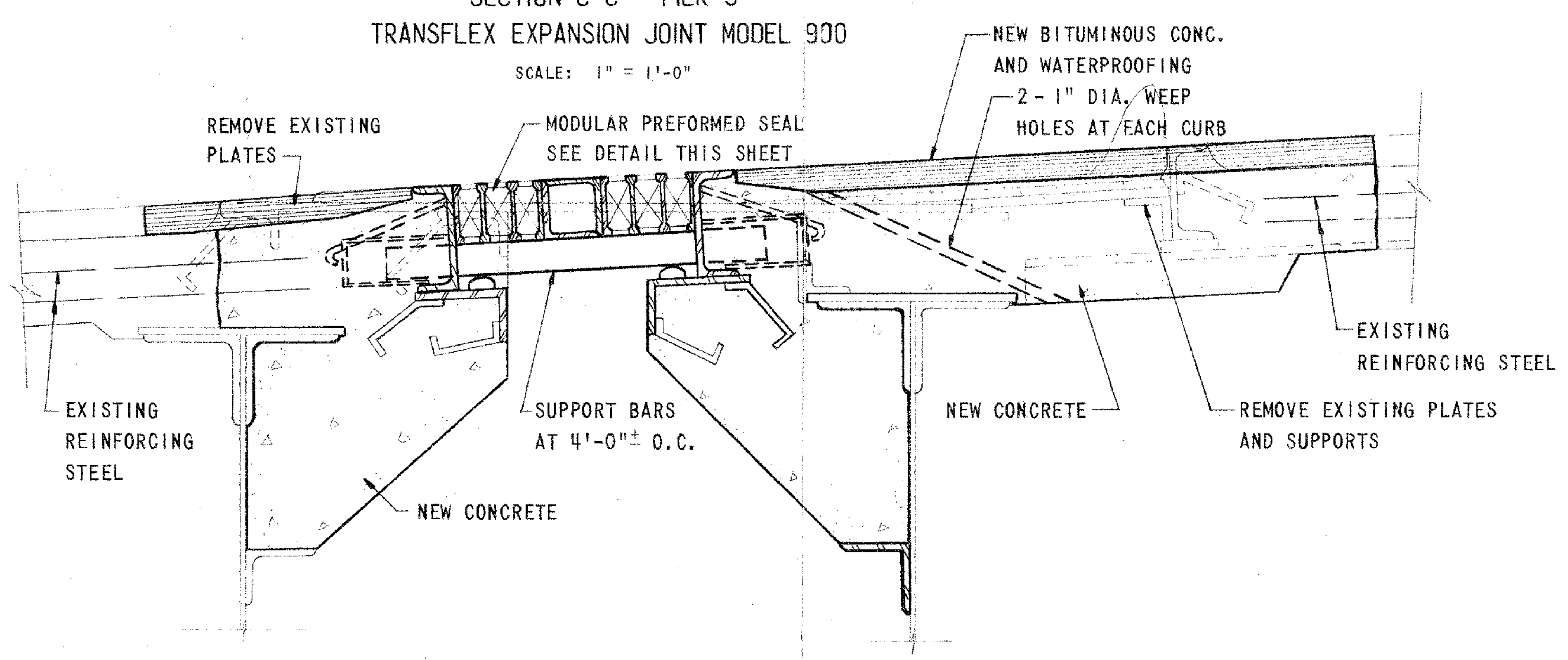
PREFORMED SEAL
SCALE: HALF SIZE



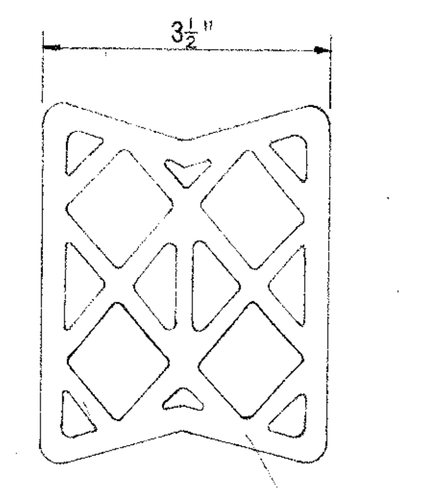
SECTION A-A - SOUTH ABUTMENT
(NORTH ABUTMENT SIMILAR)
PREFORMED SEAL
SCALE: 1" = 1'-0"



SECTION B-B - PIERS 5 AND 6
MODULAR PREFORMED SEAL
SCALE: 1" = 1'-0"



SECTION C-C - PIER 3
MODULAR PREFORMED SEAL
SCALE: 1" = 1'-0"



MODULAR PREFORMED SEAL
NOT TO SCALE:

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NEW ENGLAND DIVISION, CORPS OF ENGINEERS
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CAPE COD CANAL
BOURNE, MASS.
BOURNE HIGHWAY BRIDGE
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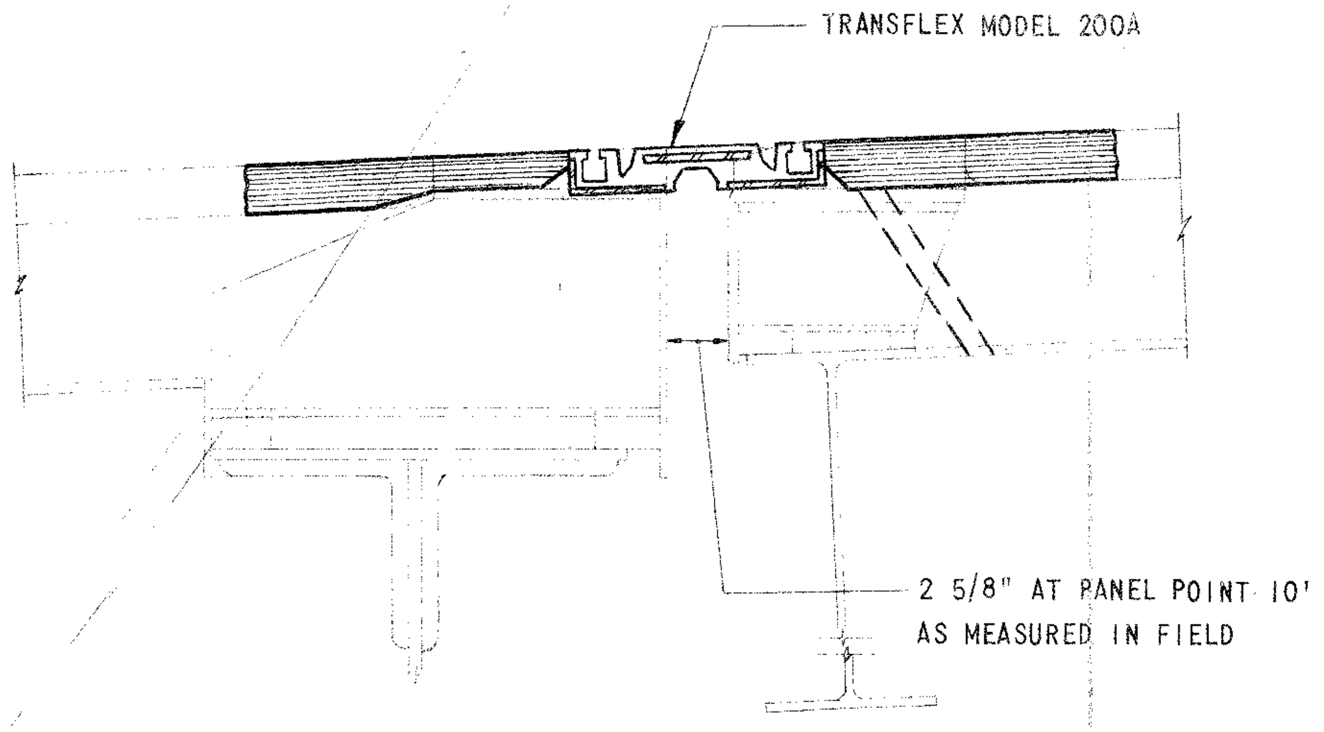
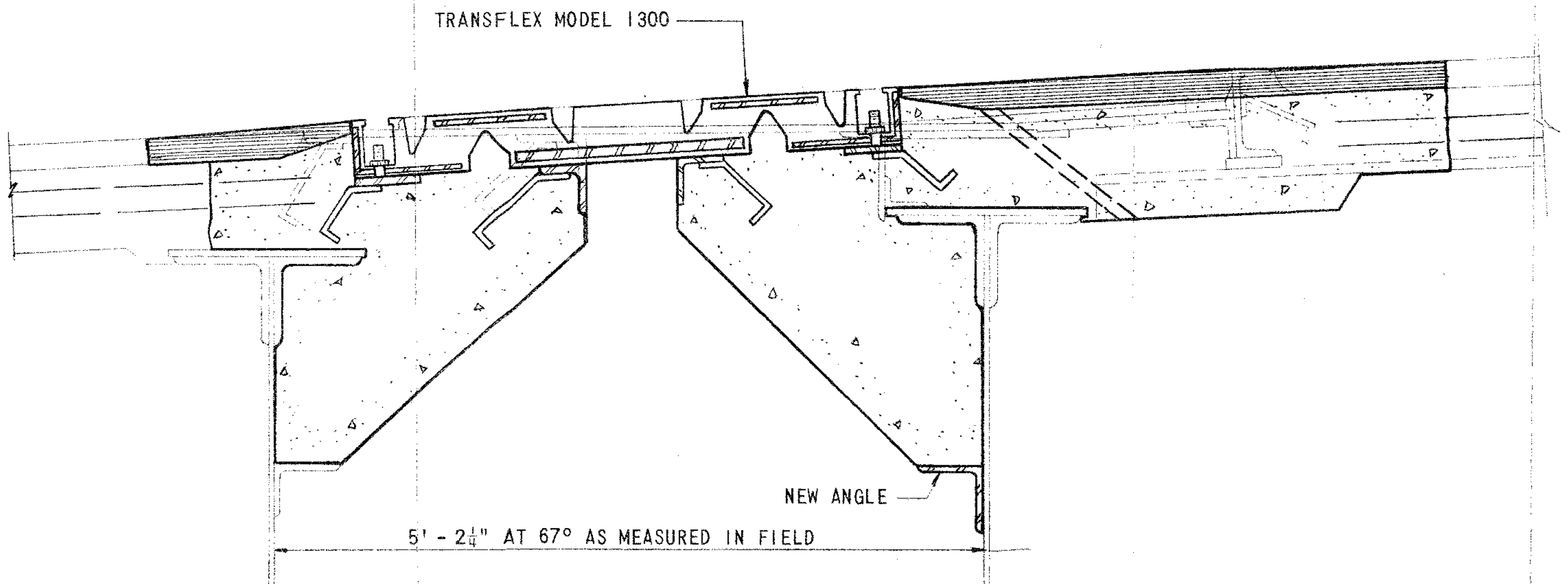
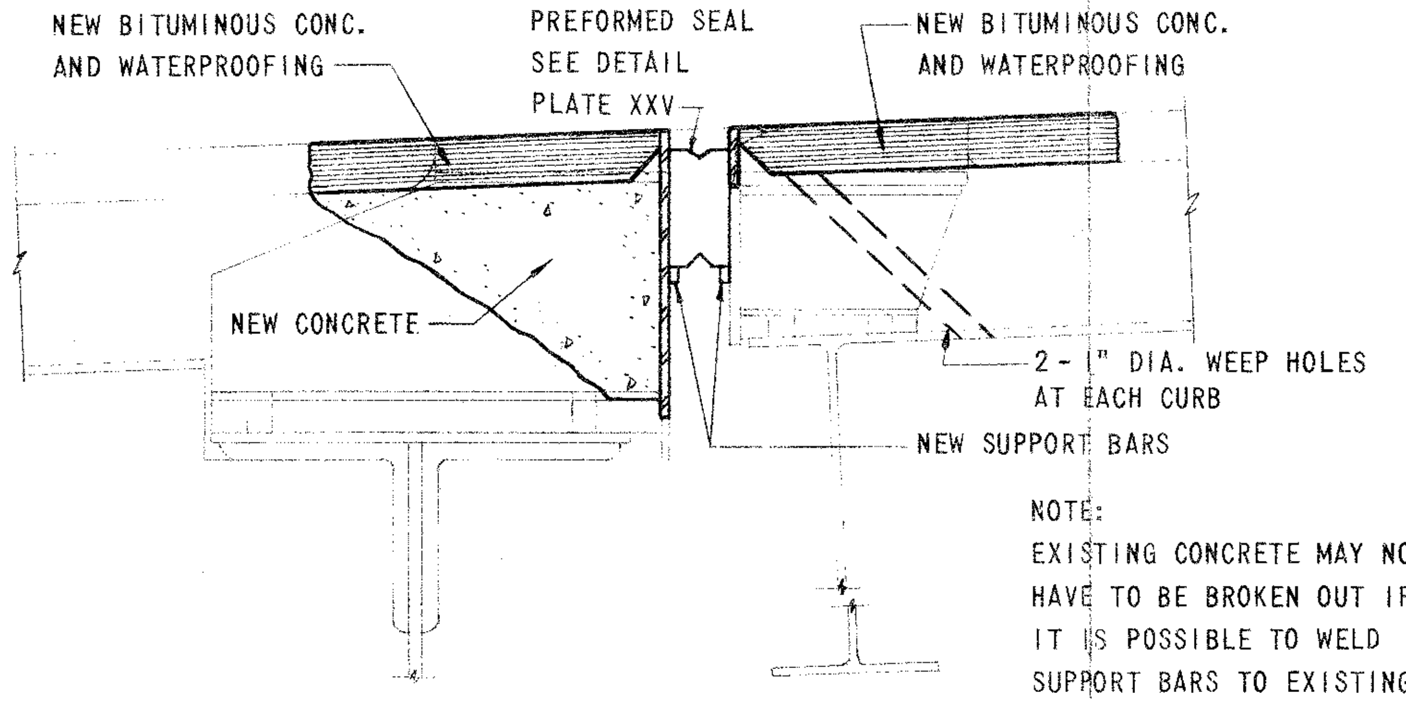


SECTION D-D - SPAN 1
AT PANEL POINTS 10 AND 10'
TRANSFLEX EXPANSION JOINT MODEL 200A
SCALE: $1\frac{1}{2}$" = 1'-0"

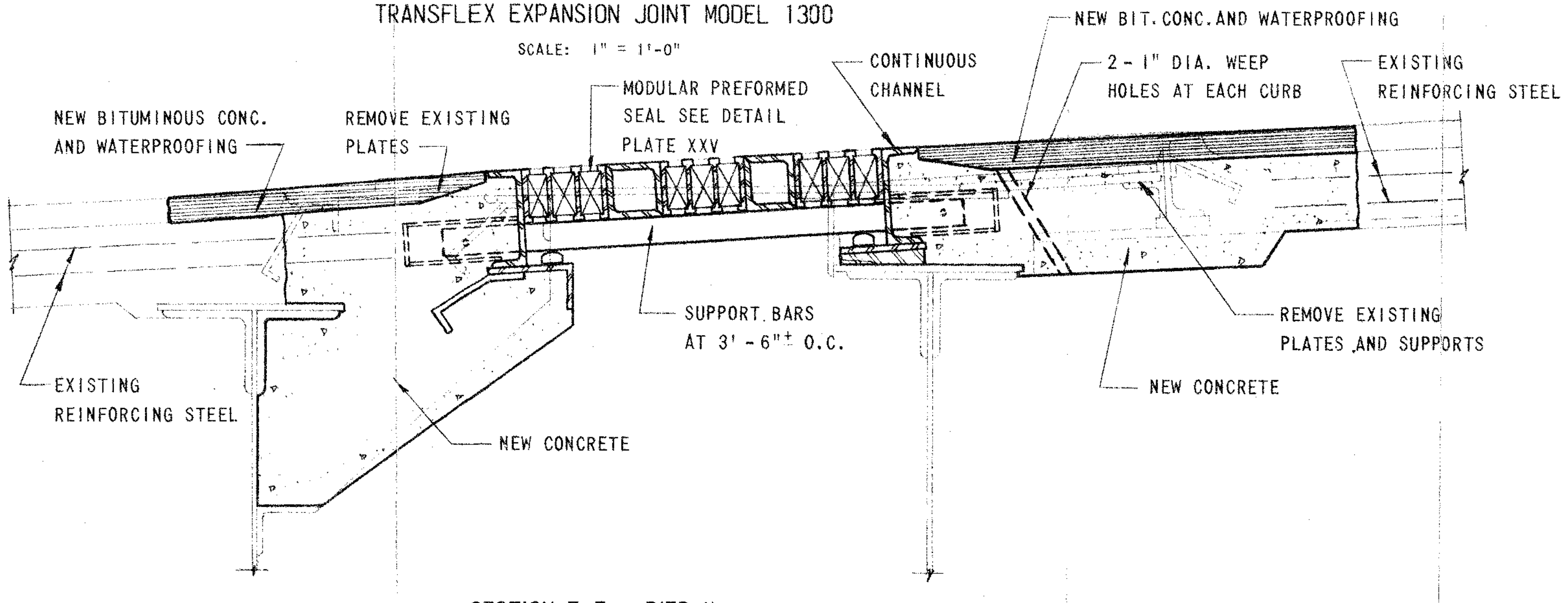


SECTION E-E - PIER 4
TRANSFLEX EXPANSION JOINT MODEL 1300
SCALE: 1" = 1'-0"



NOTE:
EXISTING CONCRETE MAY NOT HAVE TO BE BROKEN OUT IF IT IS POSSIBLE TO WELD SUPPORT BARS TO EXISTING PLATES

SECTION D-D - SPAN 1
AT PANEL POINTS 10 AND 10'
PREFORMED SEAL
SCALE: $1\frac{1}{2}$" = 1'-0"



SECTION E-E - PIER 4
MODULAR PREFORMED SEAL
SCALE: 1" = 1'-0"

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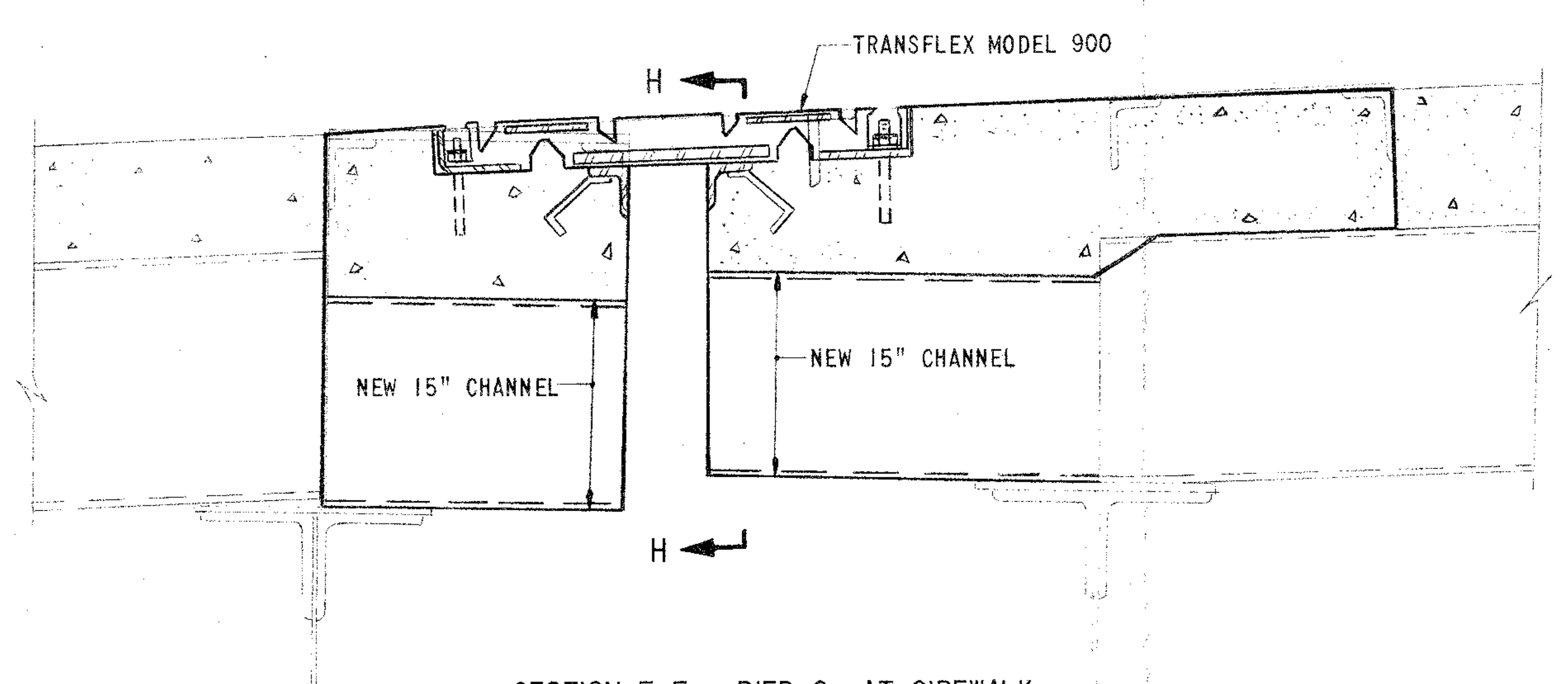
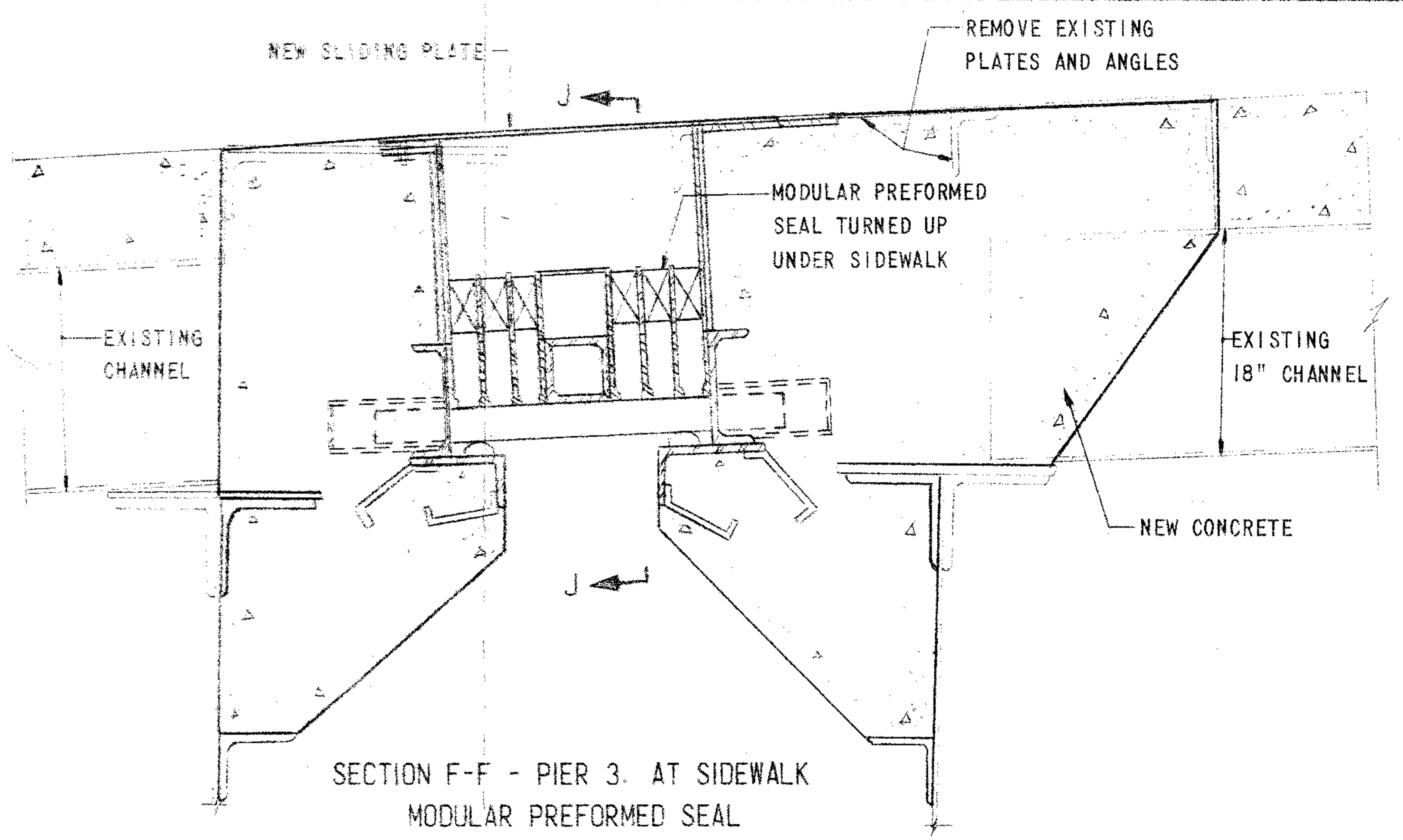
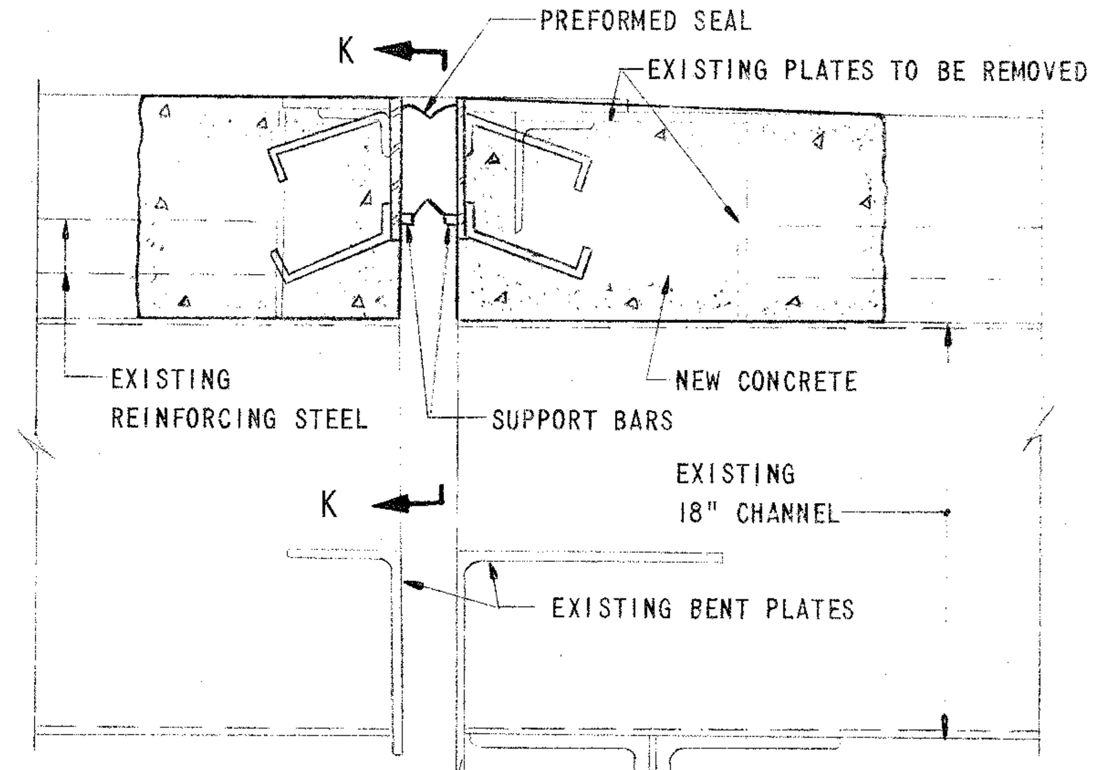


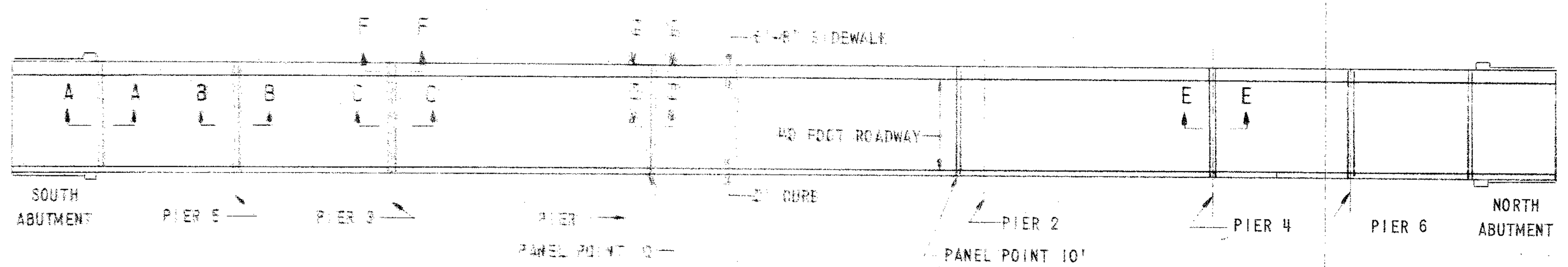
SECTION F-F - PIER 3. AT SIDEWALK
TRANSFLEX EXPANSION JOINT MODEL 900
SCALE: 1" = 1'-0"



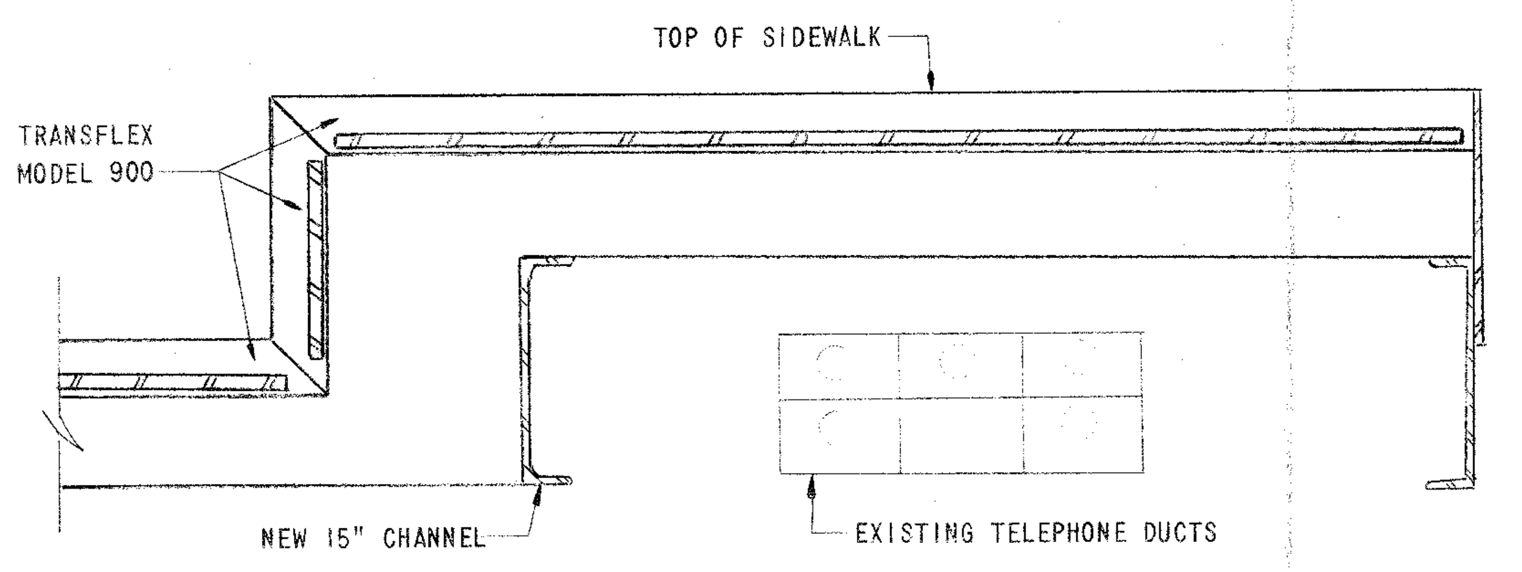
SECTION F-F - PIER 3. AT SIDEWALK
MODULAR PREFORMED SEAL
SCALE: 1" = 1'-0"



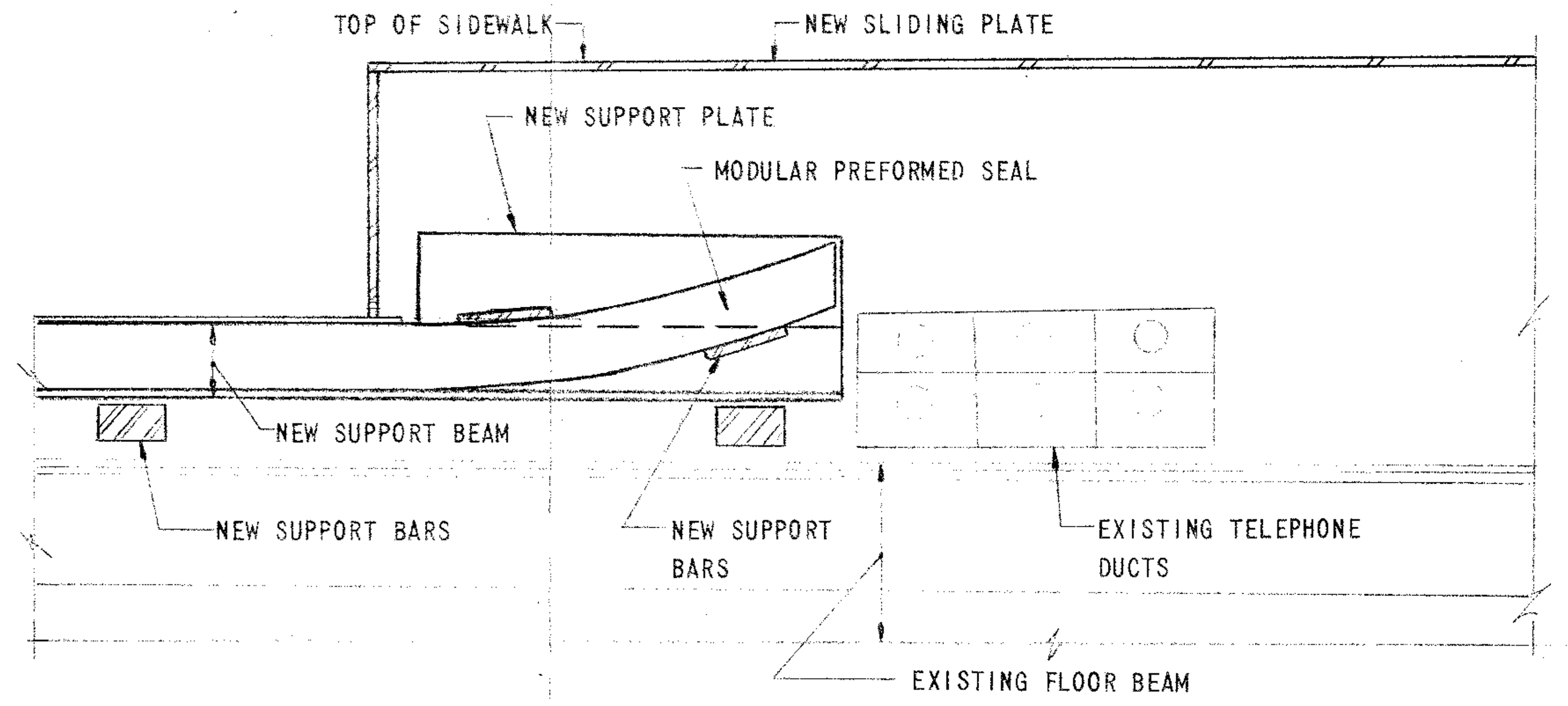
SECTION G-G - SPAN 1. AT SIDEWALK
AT PANEL POINTS 10 AND 10'
PREFORMED SEAL
SCALE: 1½" = 1'-0"



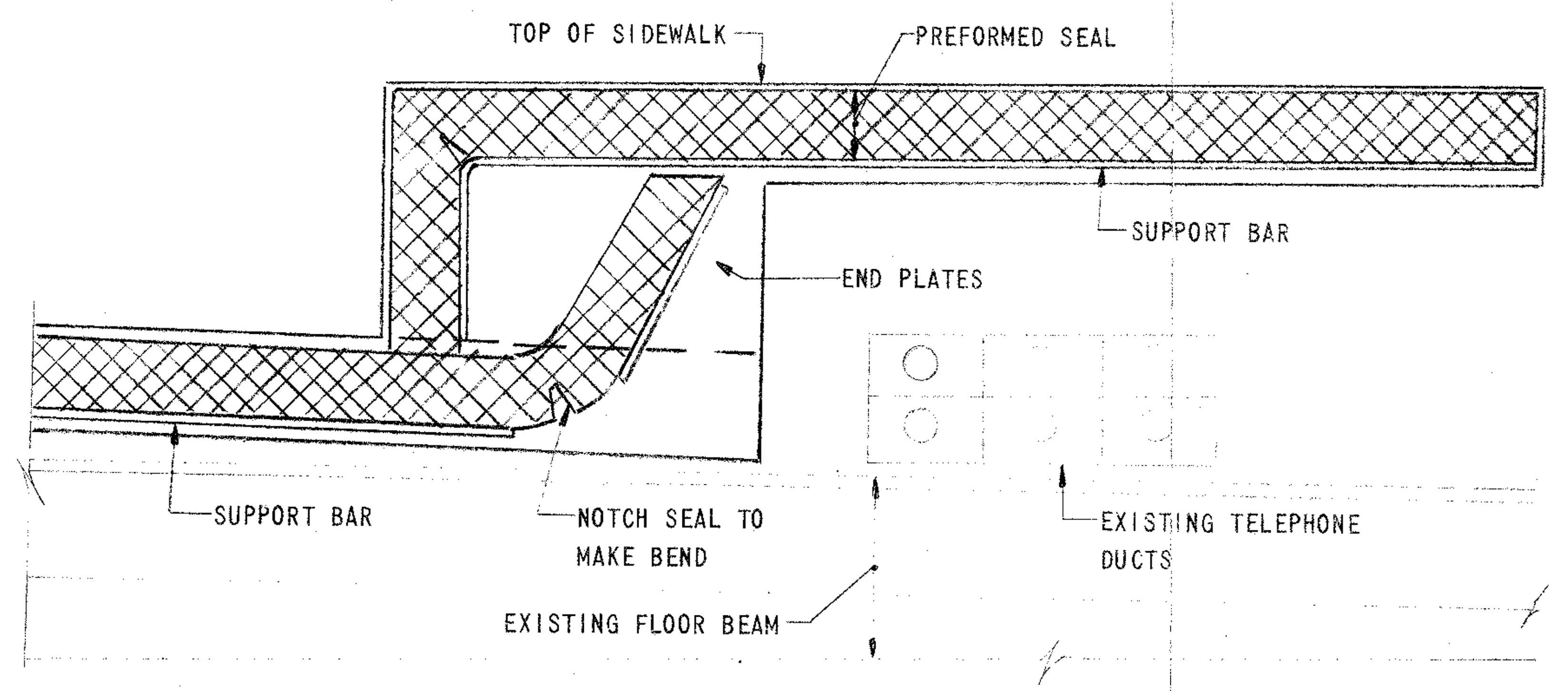
BOURNE BRIDGE
KEY PLAN TO SECTIONS



SECTION H-H
SCALE: 1" = 1'-0"



SECTION J-J
SCALE: 1" = 1'-0"



SECTION K-K
SCALE: 1" = 1'-0"

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